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MA NEW ENGLAND DIV SEP 78

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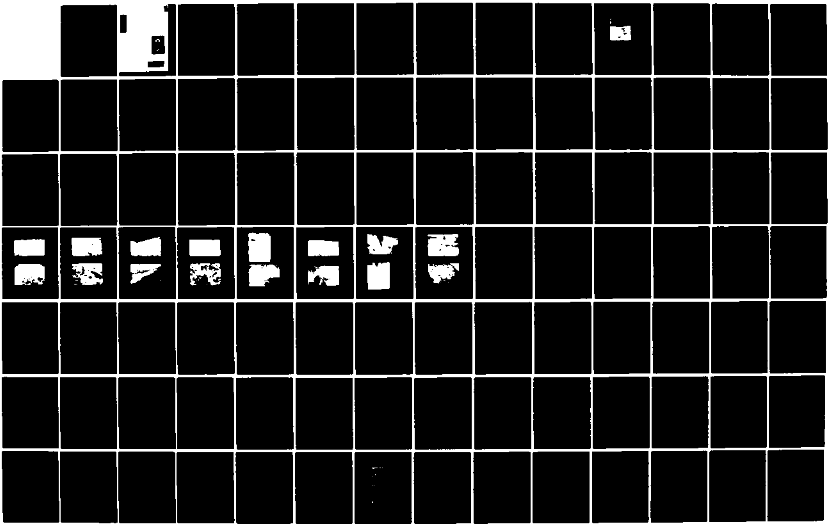

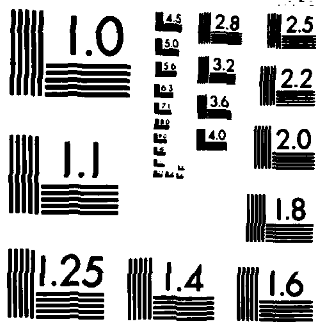
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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

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Somerset Massachusetts
Labor-in Vain Brook

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The reservoir was formed by an intermediate size dam to create an offstream water supply for the town of Somerset. The dam consists of an earth embankment about 6700 ft. long with a maximum height of 45 ft. The earth embankment is generally in excellent to good condition. Recommendations for additional investigations of the cause and extent of embankment seepage and the effect of seepage on slope stability are included in the report.

SOMERSET RESERVOIR DAM
MA 00792

MASSACHUSETTS-RHODE ISLAND COASTAL BASIN
SOMERSET, MASSACHUSETTS

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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PHASE I INVESTIGATION REPORT
NATIONAL DAM INSPECTION PROGRAM

Identification No.:	MA 00792
Name of Dam:	Somerset Reservoir
Town:	Somerset
County:	Bristol
State:	Massachusetts
Stream:	Labor-In-Vain Brook
Date of Site Visit:	19 July 1978

BRIEF ASSESSMENT

Somerset Reservoir was formed by an "intermediate" size dam build in 1965 to create an offstream water supply for the Town of Somerset. The dam consists of an earth embankment approximately 6700 ft. long with a maximum height of 45 ft. There is a small "emergency spillway" at the north end of the dam and two 20-in. outlet pipes from an intake tower in the reservoir. Somerset Reservoir is currently classified as having a "high" hazard potential in the Corps of Engineers National Inventory of dams.

Based on visual examination, the earth embankment is generally in excellent to good condition. While there was no evidence of settlement, lateral movement or other serious defects, there were indications that seepage is occurring in two areas on the downstream slope, although no actual seepage flow, boils or erosion were observed. Riprap and screened gravel bedding have been eroded by wave action in localized areas.

Hydraulic analyses indicate that the reservoir has the storage capacity to contain entering runoff from the test flood, which is based on the probable maximum flood, without overtopping the dam.

Recommendations for additional investigations of the cause and extent of embankment seepage and the effect of seepage on slope stability are included in Section 7.2.

Finally, recommendations for remedial work including clearing and mowing the downstream slope and emergency spillway and channel, repairing riprap in localized areas, replacing the abutment foundation for the access bridge to the intake tower and preparing formal plans for operation and maintenance to the dam and

and for action in the event of an emergency are described in Section 7.3.

HALEY & ALDRICH, INC.
by:

Harl Aldrich
President

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I investigations are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the test flood is based on the estimated "probable maximum flood" for the region (greatest reasonably possible storm runoff), or a fraction thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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Somerset Reservoir Dam (Spring, 1978)

PHASE I INVESTIGATION REPORT
NATIONAL DAM INSPECTION PROGRAM
SOMSERSET RESERVOIR DAM
MA 00792

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

A. Authority. Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region.

Haley & Aldrich, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Massachusetts. Authorization and notice to proceed were issued to Haley & Aldrich, Inc. under a letter dated 26 April 1978 from Colonel Ralph T. Garver, Corps of Engineers. Contract No. DACW33-78-C-0301 has been assigned by the Corps of Engineers for this work. Camp, Dresser & McKee, Inc. was retained as consultant to Haley & Aldrich, Inc. on the structural, mechanical/electrical and hydraulic/hyrologic aspects of the investigation

B. Purpose. The primary purposes of the National Dam Inspection Program are to:

1. Perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-Federal interests.

2. Encourage and prepare the states to initiate quickly effective dam safety programs for non-Federal dams.

3. To update, verify and complete the National Inventory of Dams.

1.2 PROJECT DESCRIPTION

A. Location. The Somerset Reservoir dam is located on Labor-In-Vain Brook in the Town of Somerset in Bristol County, Massachusetts, as shown on the Location Map, page viii.

B. Dam and Appurtenances. The dam at Somerset Reservoir consists of an earth embankment approximately 6500 ft. long, an intake tower for water supply and an emergency spillway, as shown on the "Site Plan Sketch" included in Appendix C-1.

The earth embankment is a homogeneous structure of "compacted glacial till and pervious fill" with a 15-in. concrete core wall set at the centerline. The core wall bears on a 3 ft.-3 in. wide footing.

The maximum height of the embankment is about 45 ft. and the crest is about 18 to 20 ft. wide. The upstream and downstream slopes are 2 horizontal to 1 vertical. There is a 6 ft. wide berm at El. 25 on the downstream slope where the embankment is high. The upstream slope is paved with dumped riprap on a screened gravel bedding. The downstream slope is covered by tall grass and weeds.

Internal drainage features include a 10-in. drain placed immediately downstream of the concrete core wall and 6-in. toe drains. The drain pipe are porous wall concrete and they discharge at two locations, Sta. 10+00 and 42+00.

The outlet works for water supply include a reinforced concrete intake tower on the reservoir side with a 44 ft. long access bridge at Sta. 12+20 and two parallel 20-in. ductile iron discharge pipes. Water may enter the intake tower through a lower level intake in the reservoir at approximately El. 18.5 or through two 16-in. by 16-in. intakes in the tower at El. 35.0 and El. 50.0.

From the intake tower, one 20-in. pipe acts as a reservoir drain by discharging at the toe of the dam into a paved gutter leading to Labor-In-Vain Brook. The second 20-in. pipe feeds the treatment plant. This pipe is under full hydrostatic pressure.

Details and sections at the outlet works are shown on a drawing by Whitman & Howard, Inc. in Appendix B-2. The location of the outlet works is shown in the "Site Sketch Plan", Appendix C-1.

A water treatment plant has been constructed immediately downstream of the outlet works.

An "emergency spillway" leading into an overflow ditch provided at the north end of the reservoir. The approach channel is basically the bed of Labor-In-Vain Brook which normally flows into the reservoir. When the water level rises, water backs up the channel through two 36-in. diameter concrete culvert pipes under relocated North St. Immediately north of North St., on the east side of the brook channel, a short trapezoidal "spillway" has been constructed through a low earth embankment. The bottom width of the spillway is about 6 ft. Once water discharges over the spillway, it flows in an easterly direction toward Broad Cove. The elevation of the "spillway" is said to be approximately 58.0.

C. Size Classification. Somerset Reservoir has an estimated maximum storage of 2700 acre-feet and the embankment has a maximum height of about 45 ft. Storage of from 1000 to 50,000 acre-feet and a height of from 40 to 100 ft. classifies the dam in the "intermediate" size category, according to guidelines established by the Corps of Engineers.

D. Hazard Classification. Somerset Reservoir is currently classified as having a "high" hazard potential in the Corps of Engineers National Inventory of Dams. Computations based on "Guidance for Estimating Downstream Dam Failure Hydrograph", included in Appendix D, confirm this classification. A failure of the embankment, depending on its location, would probably cause loss of life and extensive property damage.

For example, if the southeasterly sector of the dam failed, the houses south of St. Patrick's Cemetery and west of County Street would be very susceptible to flooding. It is estimated that a total of ten homes, the water filtration plant, and a shopping center on County Street adjacent to Whetstone Hill Road would be affected by a dam failure. In addition to the aforementioned damage to surrounding structures, it is also probable that Whetstone Hill Road and County Street culverts would be washed out. Therefore, it is recommended that the current hazard potential classification of "high" be retained.

E. Ownership. The dam and reservoir are owned by the Town of Somerset. The address of the owner is Somerset Water Department, 3249 County St., Somerset, MA 02726 (phone 617/679-2731). Mr. Joseph Gosselin, Superintendent, acted as owner representative during this investigation.

F. Operator. Mr. Joseph Gosselin, Superintendent of the Somerset Water Department, is responsible for operation and maintenance of the facility.

G. Purpose of Dam. Somerset Reservoir serves only as a water storage reservoir for the Town of Somerset. It impounds water from Labor-In-Vain Brook and water pumped from the Segreganset River at a point some four miles north of the reservoir.

H. Design and Construction History. Design of the Somerset Reservoir project was started in 1963 by the engineering firm of Whitman & Howard, Inc., Wellesley, MA. The intent was to create an offstream water supply reservoir to furnish a safe yield of 4,000,000 gallons per day for the Town of Somerset, MA. Details of the reservoir construction were published in the 31 January 1966 issue of New England Construction and summarized below.

Construction began about 1965 and was let in several contracts. One contract involved building a dam and a 15-MGD capacity low-lift pumping station on the banks of the Segreganset River. Under another contract, 19,000 linear ft. of 30-in. pipe was laid to carry water from the river to the north end of the reservoir construction site.

The site of the reservoir consisted mainly of wet fields and swamps underlain by glacial till. About 175 acres were cleared, North Street and several houses were relocated, and some 30,000 cu. yds. of soft topsoil were stripped from the area prior to construction of the embankment.

Approximately 500,000 cu. yds. of glacial till was excavated from the reservoir site and placed to form the embankment of the 6700-ft. long crescent-shaped dam. The dam was built about a vertical core wall for which 8500 cu. yds. of concrete were placed. The 36,000 cu. yds of riprap required for the upstream face was obtained from the old stone walls that abounded in the area. Salah & Pecci of Canton, MA was the contractor for construction of the dam.

I. Normal Operating Procedures. Water is pumped to the reservoir through a 30-in. pipe from the Segreganset River primarily from fall to spring. Water is taken from the reservoir for water supply by a 20-in. pipe. The reservoir can be controlled by a second parallel 20-in. pipe which serves as a reservoir drain.

There are no formal operational and maintenance procedures at Somerset Reservoir.

1.3 PERTINENT DATA

All record plans for Somerset Reservoir are on U.S.G.S. Mean Sea Level Datum (MSL).

A. Drainage Area. The drainage area at the outlet from Somerset Reservoir is 922 acres (1.44 square miles). The lake surface comprises 165 acres (17.9 percent) of this total. The topography of the watershed is coastal to rolling with an average slope of approximately 1 percent. The majority of the watershed is wooded with small sections of marshland and residential development.

B. Discharge at Damsite

1. Outlet works (conduits)............	20-in. with invert El. 14.5 at toe of dam
2. Maximum known flood at dam site..	No significant floods in area since reservoir constructed in 1965
3. Ungated spillway capacity at top of dam.........................	Not applicable
4. Ungated spillway capacity at test flood pool elevation.............	Not applicable
5. Gates spillway capacity at normal pool elevation..................	Not applicable
6. Gated spillway capacity at test flood pool elevation.............	Not applicable
7. Total spillway capacity at test flood pool elevation.............	Not applicable
8. Total project discharge at test flood pool elevation..............	56 cfs at El. 59.2

C. Elevation (ft. above MSL)

1. Top of dam........................	59.5 at center of roadway
2. Test flood pool-design surcharge...	59.2
3. Design surcharge - original design..	Unknown
4. Full flood control pool.............	56.0
5. Recreation pool....................	Not applicable
6. "Emergency Spillway" crest.......	58 (Est.)
7. Upstream portal invert diversion tunnel.........................	Not applicable
8. Streambed at centerline of dam.....	18 (Est.)
9. Maximum tailwater.................	Not applicable

D. Reservoir

1. Length of maximum pool...........	0.85 mi. (Est.)
2. Length of recreation pool..........	Not applicable
3. Length of flood control pool........	0.85 mi. (Est.)

E. Storage (acre-feet)

1. Top of dam........................	2700
2. Test flood pool....................	2650
3. Flood control pool.................	2090
4. Recreation pool....................	Not applicable
5. Spillway crest.....................	Not applicable

F. Reservoir Surface (acres)

1. Top of dam........................	182.7 (Est.)
2. Test flood pool....................	181.3 (Est.)
3. Flood-control pool................	165.0 (Est.)
4. Recreation pool....................	Not applicable
5. Spillway crest.....................	Not applicable

G. Dam

1. Type..............................	Earth (homogeneous)
2. Length............................	6500 feet
3. Height............................	45 feet max.
4. Top Width.........................	20 feet

5. Side Slopes........................ 2:1 U/S and D/S
6. Zoning............................ Probably none
7. Impervious Core.................. 15-in. concrete wall
8. Cutoff............................. None
9. Grout curtain...................... None
10. Other............................. Core wall drain and toe drains

H. Diversion and Regulating Facilities. Not applicable.

I. "Emergency Spillway" (to overflow ditch)

1. Type.............................. Vegetated trapezoidal overflow channel located north of **North Street**
2. Length............................ About 6 feet
3. Crest elevation................... Invert of overflow channel at Labor-In-Vain Brook is El. 58 (Est.)
4. Gates.............................. None
5. U/S Channel....................... 2 percent (Est.)
6. D/S Channel....................... 2 percent (Est.)

J. Regulating Outlets. Two 20-in. Class 250 mechanical joint ductile iron pipes serve as the dam's regulating outlets. One pipe acts as the reservoir drain and has an invert at El. 14.5 at the toe of the dam. The other pipe carries flow to the water treatment plant. Two sluice gates serve as the inlet control for the pipe leading to the water treatment plant and one sluice gate controls the drain. In addition, there is a gate valve at the head end of each pipe and a butterfly valve is located at the downstream end of the line to the water treatment plant.

During an impending emergency, and assuming that the reservoir's water surface elevation is at its normal level of El. 56 msl, the opening of the 20-inch drain would cause the elevation of the reservoir to fall an estimated 2 ft. (approximately 5 percent of the total water depth of 41.5 feet) in the first 24 hours. However, it should be noted that as the water surface elevation drops, the drain's capacity also decreases. Therefore, a 2 ft. drop in 24 hours is the maximum rate that could safely be expected to occur. An analysis of the brook channel downstream of the drain outlet demonstrated that it can handle this flow out of the drain.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN RECORDS

The contract drawings and specifications by Whitman & Howard, Inc. of Wellesley, MA are available and listed in Appendix B-1. However, no design calculations could be located by Whitman & Howard, Inc.

2.2 CONSTRUCTION RECORDS

The original construction contract documents are listed in Appendix B-1. Whitman & Howard, Inc. supervised construction operations, but no inspection reports or records of those operations were available.

2.3 OPERATION RECORDS

Operational records in the form of water level readings, water pumped to the reservoir and water taken for water supply are available.

There are no records of flow from the core wall drain in the dam and from embankment toe drains.

2.4 EVALUATION

A. Availability. Design and construction records are available at Whitman & Howard, Inc., 45 William Street, Wellesley, MA 02181. Operation records are available at the Somerset Water Department, 3249 County Street, Somerset, MA 02726.

B. Validity. No reason was found to question the validity of the available information.

C. Adequacy. The available data, in combination with the visual examination described in the following section, are adequate for the purpose of the Phase I investigation.

SECTION 3 - VISUAL EXAMINATION

3.1 FINDINGS

A. General. The Phase I visual examination of the Somerset Reservoir Dam was conducted on 19 July 1978.

In general, the project was found to be in excellent to good condition. A few deficiencies which require correction were noted.

A visual inspection check list is included in Appendix A and selected photographs of the project are given in Appendix C.

B. Dam. The earth embankment is generally in excellent to good condition. There was no evidence of settlement, lateral movement or other serious defects. The upstream slope which is paved with dumped cobble-boulder riprap is in good condition except for localized areas. The usual condition is shown in Photo No. 2. Except for two areas in which seepage is occurring, the downstream slope is also in good condition.

The following deficiencies were noted:

1. Riprap and screened gravel bedding have been eroded by wave action in localized areas, Photo No. 4. While repairs to the riprap are common in the vicinity of Sta. 35+00 to Sta. 50+00, Photo No. 3, other areas need attention to prevent progressive erosion.

2. Two areas of seepage on the downstream slope were noted. The first, and most important, is located in the vicinity of Sta. 9+00, Photos No. 5 and 6. The top of the wet area, in which cattails are growing, is approximately 8 ft. vertically above the berm. Water is ponded on the berm and the wet area extends to the toe of the embankment and beyond. No actual seepage flow, boils or erosion were observed.

   The second area of seepage, and the only other location where the toe of the 6500 ft. embankment was wet, occurs about Sta. 14+50. Again, no flow was observed.

3. Knee-high to waist-high grass, weeds, brush and occasional saplings cover the downstream slope, Photo No. 7. Grass

is mowed on the dam crest only, Photo No. 1. Because of the grass, the slope was very difficult to examine carefully.

4. Numerous animal holes, probably woodchuck, were observed on the downstream slope. Since the dam has a concrete core wall, the presence of these holes is not considered significant.

The core wall drains and toe drains discharge at two locations beyond the downstream toe of the dam, Sta. 10+00 and Sta. 42+00. At each location, a 10-in. porous wall concrete pipe flanked by two 6-in. porous wall pipes discharge at a stone headwall. All six pipes were flowing. Photo No. 8 shows the three pipes at Sta. 10+00 after weeds and grass were cleared. Photos No. 9 and 10 show the submerged drain pipes at Sta. 42+00 and a wet swampy area which occurs downstream of the point of discharge. Water discharging from the pipes was clear and cold except for floating flecks of rust-brown organic matter. This material has stained the invert of the pipe and the stream beds as shown on the photographs.

C. Appurtenant Structures. Concrete for the intake tower was found to be in excellent condition. The access bridge to the intake, Photo No. 11, was found to have substantial compressive stresses locked into the top and bottom chords of the steel trusses. This has resulted in a small but noticeable bow to the structure. The abutment has moved downward and outward since the original construction. The abutment received a concrete mortar facing, the front and sides of which have further cracked and moved downward and outward, Photo No. 12. This mortar facing, when installed, locked the bridge to the abutment, resulting in the aforementioned compressive force.

The hand-operated valve to the 20-in. reservoir drain, located at the intake tower, was operated and appears to be in satisfactory condition.

Photo No. 13 shows the outlet of the 30-in. diameter intake pipe, located at the north end of the reservoir, which delivers water to Somerset Reservoir from the Segreganset River.

The only outlet from the reservoir, other than the reservoir drain and intake pipe to the water treatment plant, is the emergency spillway described in Section 1.2B. While the approach channel and

culverts were in satisfactory condition, Photos No. 14 and 15, the spillway and discharge channel are overgrown with brush and small trees. In fact, the spillway is totally obscured by vegetation, Photo No. 16. Debris in the form of discarded pipes was found in the channel below the "spillway".

D. Reservoir Area. The area around the west side of Somerset Reservoir is generally wooded with relatively flat slopes. There are no conditions which would lead to a significant increase in sediment load to the reservoir or landslides which would cause waves to overtop the dam.

E. Downstream Channel. With construction of Somerset Reservoir, Labor-In-Vain Brook carries discharge from the reservoir drain when opened, and flow from underdrains at Sta. 10+00. The channel is more than adequate to carry these flows.

The channel below the emergency spillway has been discussed in Section 3.1C.

3.2 EVALUATION

While the Somerset Reservoir dam and appurtenant structures are generally well maintained and in excellent to good condition, there are a few deficiencies which require correction. Nevertheless, there appears to be no significant potential for failure of the dam.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

In general, there are no formal operation and maintenance procedures for Somerset Reservoir except for regulation of water levels. Water level in the reservoir is recorded. When the level approaches normal pool level or just above, the reservoir drain is opened to maintain the water level at this elevation. Since the structure is primarily pump storage, the water level can be controlled by the amount of pumping to the reservoir and by the control exercised through the use of the reservoir drain.

4.2 MAINTENANCE OF DAM

There are no known procedures to require inspection and routine maintenance of the dam and emergency spillway. Except for occasional mowing of grass which covers the embankment crest and repairs to riprap when and where localized erosion occurs, the dam does not receive regular maintenance.

4.3 MAINTENANCE OF OPERATING FACILITIES

The visual inspection indicated that the gates and other operating facilities are well maintained.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT

There is no emergency preparedness plan or warning system in effect for this dam.

4.5 EVALUATION

The emergency spillway requires maintenance to insure its operation as previously mentioned in Section 3. Other areas appear to be well maintained including the operating facilities. A warning system should be formalized for the structure.

For a high hazard structure of this importance, operation and maintenance procedures for Somerset Reservoir should be formalized to assure periodic inspection and continued good maintenance and satisfactory operation. An emergency preparedness plan should also be adopted.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

A. Design Data. A set of plans entitled "Proposed Surface Water Supply Reservoir - Somerset, Mass." bearing the date of September 1964 were the basis for the construction of this facility. This reservoir was constructed in order to solve the water supply problem for the town. It was designed for a safe yield of 4,000,000 gallons of water per day. However, no hydraulic design data were found for Somerset Reservoir Dam.

The recommended test flood for the size (intermediate) and hazard potential classification (high) of this dam is the probable maximum flood (PMF).

B. Experience Data. Because of the small magnitude of the drainage area, the "SCS-TP-149, Method for Estimating Volume and Rate of Runoff in Small Watersheds" was used as a guide for determining the inflow hydrograph into Somerset Reservoir for the PMF. The PMF was based on a 25-in. rainfall in 6 hours. The peak inflow rate generated from the entire watershed was 5160 cfs.

However, since the volume of runoff entering the reservoir from the northern section of the watershed, upstream of North Street, is controlled by the twin 36-in. culverts, the majority of the flow from the northern portion of the watershed follows the course of the overflow ditch and eventually empties into Broad Cove. At the time of the peak of the storm, only approximately 11 percent (440 cfs) of the flow from the northern drainage area would enter the reservoir via the twin 36-in. R.C. pipes and an estimated 89 percent (3690 cfs) would exit via the overflow ditch.

The peak inflow rate generated from the portion of the watershed which drains immediately into Somerset Reservoir, approximately 362 acres, was approximately 2980 cfs. (This value includes the effect of rainfall directly on the reservoir.) When routed through the reservoir, the value of 2980 cfs was reduced to a mere 56 cfs (capacity for the 20-in. drain pipe) as the major portion of the inflow was stored, resulting in maximum reservoir level of El. 59.2, only 0.3 ft. below the crown elevation of the access road along the entire dam.

C. Visual Observations. The inspection revealed that no significant modifications have been made to the inlet or outlet works since the construction of the dam. It was noted that a few of the

stones from the downstream headwell of the twin 36-in. RC culvert beneath North Street had become loosened and fallen into the pipe outlet, thereby partially obstructing low flows. The stones would probably be carried through the culvert by high brook flows. It was also noted that the upstream end of this culvert was partially blocked by a combination of stones and bags of cement, the remains of a small diversion structure apparently used during recent sewer construction to divert Labor-In-Vain Brook flows down the overflow ditch.

The overflow ditch was noted to have a typical section in the upstream reaches with a base width of 5 to 6 ft., a top width of 12 to 14 ft. and side slopes of one vertical to one and one-half horizontal. It was also noted to be severely overgrown with weeds, brush and small bushes. The slope of the overflow ditch is about 0.012 from Labor-In-Vain Brook to a point about 900 ft. to the east, where it increases sharply by dropping 30 ft. in the next 350 ft. before becoming obscured in a swampy area.

Downstream of the dam, Labor-In-Vain Brook reforms with the interception of flow from the toe drains of the dam, as well as runoff diverted away from the reservoir during dam construction from the area immediately adjacent to, and north of, Whetstone Hill Road. Flows are conveyed past the water filtration plant site in a well maintained grass-lined ditch, after which they are conducted through culverts beneath Whetstone Hill Road (4-ft. wide by 3-ft. high stone) and County Street (36-in. R.C. pipe). About 1,000 ft. downstream of County Street, Labor-In-Vain Brook enters a vast tidal marsh over 40 acres in extent before emptying into Taunton River tidewater at Riverside Avenue.

D. Overtopping Potential. A rating curve for the overflow ditch and twin 36-in. culverts beneath North Street was developed, and the capacity of the 20-in. ductile iron reservoir drain was determined to vary from 53 cfs to 56 cfs for reservoir levels between El. 56 and at the top of the dam, El. 59.5. A study of the inflow from the watershed directly tributary to the reservoir downstream of North Street showed that using a test flood equivalent to the PMF and routing same, results in a maximum reservoir level of El. 59.2, 0.3 ft. below crown of peripheral road, with the major portion of the storm flow through the reservoir drain at a maximimum rate of 56 cfs. The reservoir level is also influenced by the admission of the excess storm flows from the upper portion of the watershed which cannot be adequately conveyed by the over-

flow ditch. These flows enter the reservoir via the twin 36-in. culvert beneath North Street, thus causing the reservoir level to rise above El. 59.2, but only by an insignificant amount.

The events that would follow an overtopping of the dam would be greatly influenced by the location of the breach. The southeasterly sector of the dam is probably the most vulnerable because it has the greatest fetch from a northwest wind. Although houses along the westerly side of Country Street north of St. Patricks Cemetery would likely avoid damage, houses south of the cemetery and west of County Street, particularly those new houses built during the past 10 to 12 years in the hollow adjacent to the dam, would be very susceptible to flooding from a dike failure. It is estimated that a total of ten homes plus the water filtration plant and a shopping center on County Street adjacent to Whetstone Hill Road would be affected by a failure of the dam.

E. Evaluation. Passage of flood flows via the "emergency spillway" to the swampy area north of North Street and eventually into Broad Cove should cause little or no damage, as the area remains remote from development.

Passage of the estimated flows from a dam failure will, in addition to the flooding described in the above Section D, probably cause minor and basement flooding to those homes around the periphery of the marsh near Riverside Avenue.

Because of the foregoing flood damage potential, a failure of this dam could result in extensive downstream damage as well as the potential for loss of life in some of the affected homes and buildings. However, as shown by the calculations in Appendix D, the dam can handle the test flood which is based on the PMF.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

A. Visual Observations. There was no visual evidence of embankment structural instability during the site examination on 19 July 1978. No erosion or piping were observed where seepage occurs on the downstream slope. Therefore, seepage is not considered to pose an immediate hazard to slope stability.

There is no other structure at Somerset Reservoir whose failure would endanger the dam, since there is no concrete or masonry spillway.

B. Design and Construction Data. Drawings by Whitman & Howard, Inc. are available which show the design cross-sections for the earth embankment, Appendix B-2. However, no design criteria for embankment stability or calculations are available. Furthermore, there are no construction records available which define soil properties.

The embankment is believed to have been constructed primarily of glacial till. With slopes of 2 horizontal and 1 vertical, and with provision for a central core wall and drains, the downstream slope can be expected to be adequately stable in the absence of significant seepage under static loading conditions.

C. Operating Records. There are no records of embankment settlement, lateral movement, pore water pressures or other information from field instrumentation.

D. Post-Construction Changes. There have been no known structural changes to the earth embankment since its construction in 1965.

E. Seismic Stability. Somerset Reservoir Dam is located in Seismic Zone 2, and in accordance with recommended Phase I guidelines does not warrent seismic analyses.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT

A. Condition. The visual examination of Somerset Reservoir Dam revealed that the project was generally in good to excellent condition. There were no signs of failure or conditions which would warrant urgent remedial treatment. However, some maintenance is required and an investigation of wet areas on the downstream slope should be undertaken.

Based on the results of computations included in Appendix D, and described in Section 5.1B, the reservoir has the storage capacity to contain runoff from the test flood, based on the probable maximum flood, without overtopping the dam.

B. Adequacy of Information. The data available concerning the design and construction of the dam are adequate for a Phase I Investigation when supplemented by field observations.

C. Urgency. The recommendations for additional investigations and remedial measures outlined in Sections 7.2 and 7.3 respectively, should be undertaken by the Town of Somerset within 24 months after receipt of this Phase I Inspection Report.

D. Need for Additional Investigation. Additional investigations are required, as outlined in Section 7.2.

7.2 RECOMMENDATIONS

It is recommended that the Town of Somerset engage a registered professional engineer experienced in dam design to undertake an investigation to determine the extent and cause of seepage noted on the downstream slope of the embankment in the vicinity of Sta. 9+00 and 14+50, especially the former. The effect of seepage on slope stability should also be assessed.

It is further recommended that the Town of Somerset engage a registered professional engineer to investigate and propose corrective action for the access bridge abutment foundation. The abutment has moved downslope and introduced additional stresses into the access bridge.

7.3 REMEDIAL MEASURES

A. Alternatives. Not applicable.

B. Operating and Maintenance Procedures. The following remedial work should be undertaken by the Town of Somerset to correct deficiencies noted during the visual examination.

1. Clear the downstream embankment slope by mowing tall grass, weeds, brush and small saplings. Unless the embankment is mowed periodically, trees will become established which are undesirable as they reach maturity.

2. Repair riprap in localized areas where failure has occurred by erosion from wave action.

3. Clear the emergency spillway and channel immediately downstream of all brush and trees, and mow the area to allow free flow of water should discharge occur. Clear boulders and other debris from concrete culvert pipes below North St.

In order to provide for long-term operation and maintenance of the dam and for action in the event of an emergency, the Town of Somerset should also:

1. Prepare a formal program to periodically inspect the project and to provide for routine maintenance.

2. Develop a formal emergency preparedness plan and warning system, in cooperation with local civil defense and police officials. This plan should include the provision that the 20-in. discharge conduit described in Section 1.3J be manned and operated when the reservoir reaches El. 57 MSL.

APPENDIX A
INSPECTION TEAM ORGANIZATION AND CHECK LIST

	Page No.
VISUAL INSPECTION PARTY ORGANIZATION	1
VISUAL INSPECTION CHECK LIST	
Dam Embankment	2
Outlet Works - Intake Channel and Intake Structure	3
Outlet Works - Emergency Spillway, Approach and Discharge Channels	4

VISUAL INSPECTION PARTY ORGANIZATION

NATIONAL DAM INSPECTION PROGRAM

Dam: Somerset Reservoir

Date: 19 July 1978

Time: 0900-1445

Weather: Clear and Hot

Water Surface Elevation Upstream: 52.6 MSL
(41 in. below full pond)

Stream Flow: Not applicable

Inspection Party:

Harl P. Aldrich, Jr. Haley & Aldrich, Inc.	- Soils/Geology
Roger H. Wood Camp, Dresser & McKee, Inc.	- Structural
Charles E. Fuller Camp, Dresser & McKee, Inc.	- Hydraulic/Hydrologic
Charles L. Loveridge Camp, Dresser & McKee, Inc.	- Mechanical/Electrical

Present During Inspection:
Joseph Gosselin, Superintendent, Somerset WaterDepartment

VISUAL INSPECTION CHECK LIST
NATIONAL DAM INSPECTION PROGRAM

DAM: Somerset Reservoir DATE: 19 July 78

AREA EVALUATED	CONDITION
DAM EMBANKMENT	
Crest Elevation	59.5 (M.S.L. Datum)
Current Pool Elevation	52.6
Maximum Impoundment to Date	El. 57.0 (25 March 1972)
Surface Cracks	None observed (but very difficult to see bare ground)
Pavement Condition	No pavement, mowed width of top of embankment about 17 ft.
Movement or Settlement of Crest	No observed
Lateral Movement	None observed
Vertical Alignment	Good
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	Satisfactory (no concrete structures except footing for bridge to intake structure, see below)
Indications of Movement of Structural Items on Slopes	Pier and footing for bridge has moved (See Photos). No other structural items on visible slopes.
Trespassing on Slopes	Frequent, no restrictions
Animal Burrows in Embankment	Numerous on downstream slope
Vegetation on Embankment	Knee-high to waist-high grass, weeds, brush and occasional saplings. Grass mowed on dam crest only.
Sloughing or Erosion of Slopes or Abutments	None of any significance observed, but surface difficult to examine.
Rock Slope Protection - Riprap Failures	Generally good, a few local failures near top where crushed stone bedding has been exposed. Riprap is dumped stone, typically 2-man size stones and smaller, minor weathering
Unusual Movement or Cracking at or near Toes	None observed. Again, difficult to examine because of vegetation
Unusual Embankment or Downstream Seepage	Seepage occurs from embankment at about Sta. 9+00, starting about 8 ft. vertically above berm; cattails in area; area wet with water ponded but no flowing water observed. At about Sta. 14+50, also wet with cattails at berm. No other seepage observed.

VISUAL INSPECTION CHECK LIST
NATIONAL DAM INSPECTION PROGRAM

DAM: Somerset Reservoir DATE: 19 July 78

AREA EVALUATED	CONDITION
Piping or Boils	None observed.
Foundation Drainage Features	6 to 10-inch drain downstream of core wall and 6-inch toe drains, flowing. (See report and photos)
Toe Drains	6-inch porous wall concrete pipe to outlet headwalls at Sta. 10+00 and Sta. 42+00.
Instrumentation Systems	None, no field measurements.
OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE	
a. Approach Channel	Not applicable
b. Intake Structure	
Tower	Concrete in excellent condition
Access Bridge	One dent in grating, entire bridge has bow due to locked shore bearings; abutment concrete encases lower bracing
Bridge Abutment	Original abutment appears to have settled and moved towards water; encased with mortar at later date locking abutment to bridge; sides and front have broken loose, settled and moved towards water; (See photos)
c. Mechanical and Electrical	(No electrical)
Service Gates	Good condition; hand-operated gates operable
d. Outlet Channel	
Channel Bottom	Rust in channel, probably due to discharge from internal drains in embankment; heavily grassed on sides

FILE NO. 4160

HALEY & ALDRICH, INC.
CAMBRIDGE, MASSACHUSETTS

VISUAL INSPECTION CHECK LIST
NATIONAL DAM INSPECTION PROGRAM

DAM: Somerset Reservoir DATE: 19 July 78

AREA EVALUATED	CONDITION
OUTLET WORKS - EMERGENCY SPILLWAY, APPROACH AND DISCHARGE CHANNELS	
a. Approach Channel	
General Condition	Satisfactory - channel is bed of brook which crosses relocated North St. at north end of dam
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	None, brush only
Floor of Approach Channel	Heavy grass, some brush and debris
b. Weir and Training Walls	Not applicable. (Emergency spillway is a short trapezoidal depression in an earth fill placed east of the Labor-In-Vain Brook channel, immediately north of North St. Spillway opening is substantially blocked by tall grass, brush and small trees.)
c. Discharge Channel	
General Condition	Fair, but channel is ill-defined and difficult to find and examine
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	Heavy brush, small trees, tall grass
Floor of Channel	Tall grass, brush overground

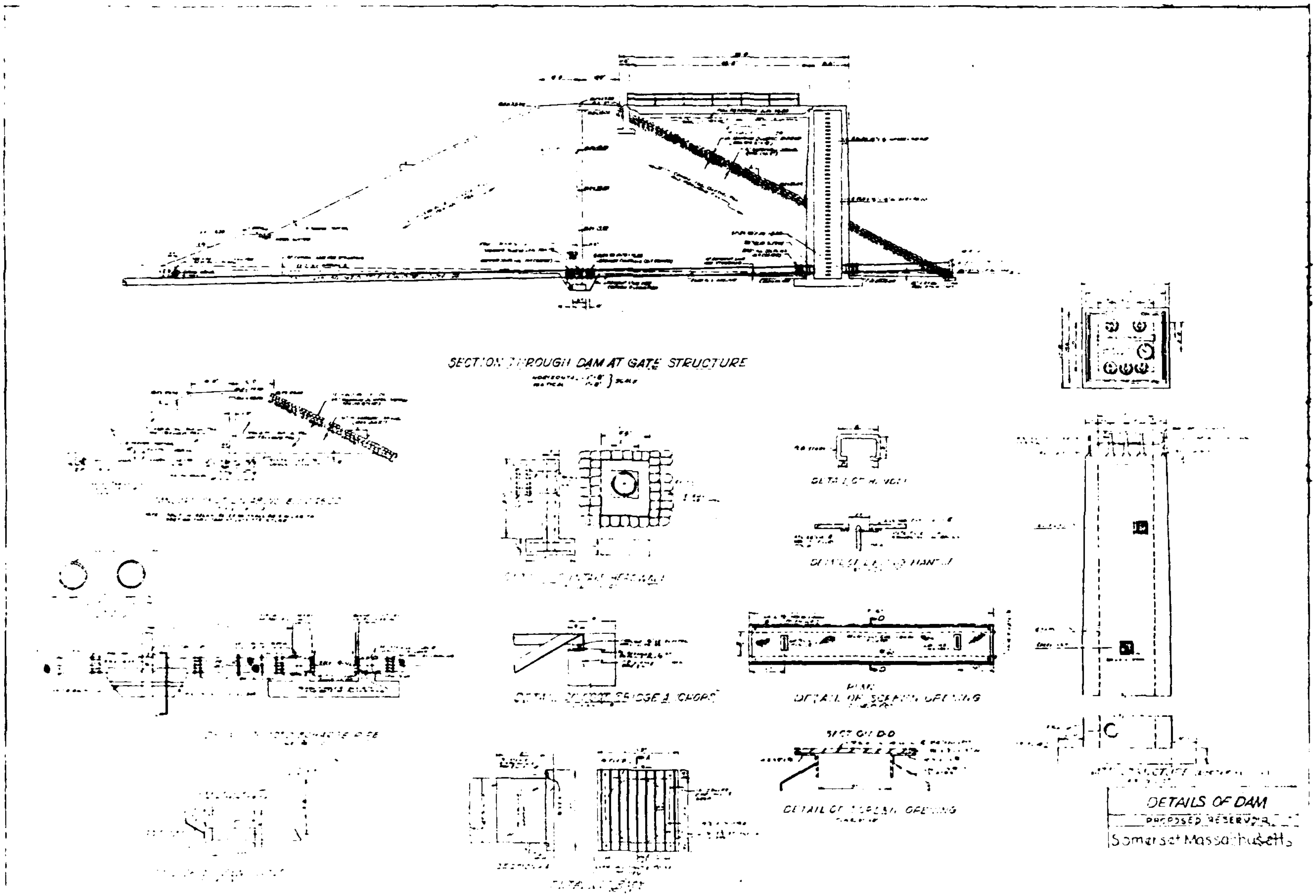
APPENDIX B
LIST OF AVAILABLE DOCUMENTS AND PRIOR INSPECTION REPORTS

	Page No.
LIST OF AVAILABLE DOCUMENTS	1
"Details of Dam", Sheet 6 of "Proposed Surface Water Reservoir, Somerset, Massachusetts", Whitman & Howard, Inc., Engineers, Boston, MA, September 1964	2
PRIOR INSPECTION REPORTS	

Date	By	Page No.
22 March 1968	Hayden, Harding & Buchanan, Inc., Boston, MA	3
27 July 1970	Universal Engineering Corp., Boston, MA	4

LIST OF AVAILABLE DOCUMENTS
SOMERSET RESERVOIR DAM

DOCUMENT	CONTENTS	LOCATION
"Proposed Surface Water Reservoir, Somerset, Massachusetts", Whitman & Howard, Inc., Engineers, Boston, MA, September 1964	Seven contract drawings entitled Locus, Plan of Dam (3 pages), Profile of Dam and Details of Dam (2 pages)	Whitman & Howard, Inc. 45 William Street, Wellesley, MA 02181
"Specifications for Constructing Storage Reservoir and Dam", Whitman & Howard, Inc., Engineers, Boston, MA, January 1965	Contractual agreement; general soil gradation, placement and compaction requirements	Whitman & Howard, Inc. 45 William Street, Wellesley, MA 02181



SECTION THROUGH DAM AT GATE STRUCTURE
PLAN
DETAIL OF SCREEN OPENING
SECT ON D-D
DETAIL OF SCREEN OPENING
DETAILS OF DAM
PROPOSED RESERVOIR
Somerset Massachusetts

Somerset 6-3-273-1

BRISTOL COUNTY, MASS.
INSPECTION REPORT & DATA FOR DAMS

PREPARED FOR THE BRISTOL COUNTY COMMISSIONERS
BY HAYDEN, HARDING & BUCHANAN, INC., BOSTON, MASS.

Dam No. So-1	Town: Somerset
Stream: [illegible]	Pond:
Date:	By:

Owner: Town of Somerset
His Address: Water Department
Function of Dam: Storage reservoir dam
Location & Access: North of Whetstone Hill Rd. & east of County St. (Rte 138)
USGS Quad. Somerset Lat. 41°45'40" Long. 71°09'20"

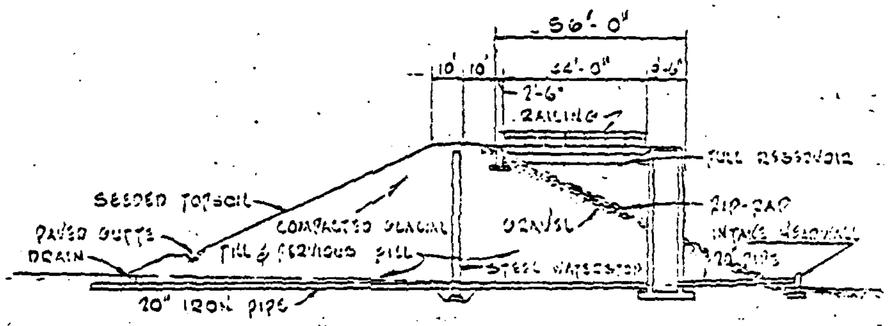
CONDITION RATING
Structural:
Hydraulic:
General:
PRIORITY: 5

Drainage Area: 1.5 sq.mi.; Ponds: ac.; Res.@dam: ac.
Character of D.A.:
Estimated Discharge Capacity:

KINNISON-COLBY FLOODS
Minor: cfs
Major: cfs
Rare: cfs
Maximum: cfs

General Description of Dam and Discharge Control: Earth dam with concrete core constructed 1935.

Sketch (Not to Scale):



SECTION THROUGH DAM AT GATE STRUCTURE

Remarks and Recommendations:

Re-inspection

Date	By	Comment
3-22-80	[illegible]	No problem water 9" high - Supt.

Dam No. So-1

BRISTOL COUNTY, MASS. INSPECTION REPORT FOR DAMS PREPARED FOR THE BRISTOL COUNTY COMMISSIONERS BY UNIVERSAL ENGINEERING CORP, BOSTON, MASS.	DAM NO. So. - 1 TOWN: Somerset

INSPECTION DATE	REMARKS & RECOMMENDATIONS
7-27-70	The gate structure is in excellent condition. The level of the pond is approximately 5 feet below the catwalk. The abutment for the catwalk has been seriously damaged apparently due to thermal expansion. The abutment backwall should be repaired, and anchor bolts reset as a safety precaution.

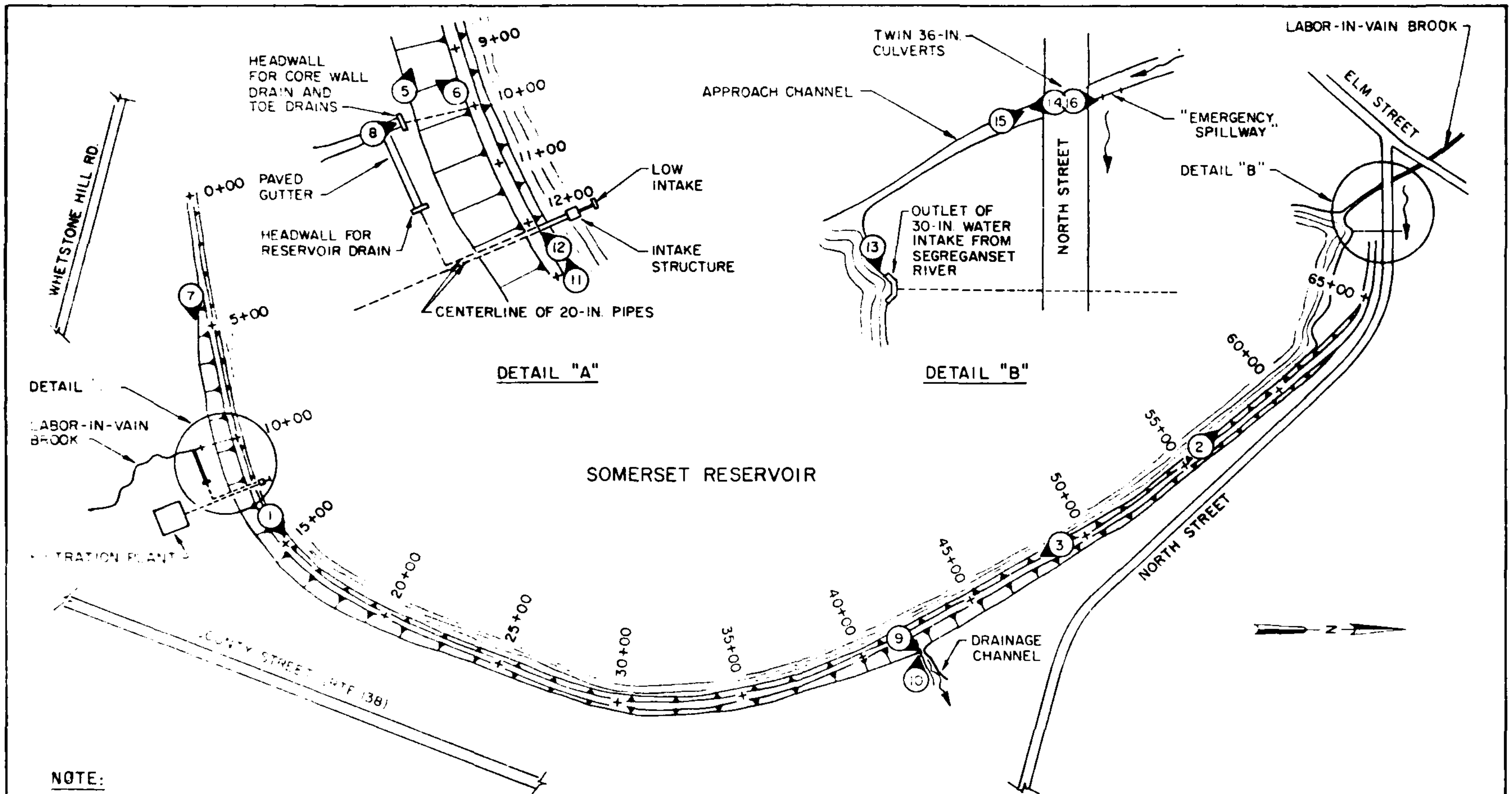
Supplement to original report and data by Hayden, Harding & Buchanan, Inc. DAM NO So. 1

APPENDIX C
SELECTED PHOTOGRAPHS OF PROJECT

	Page No.
LOCATION PLAN	
Site Plan Sketch	1

PHOTOGRAPHS

No.	Title	Roll	Frame	Page No.
1.	Crest of Dam and Upstream Slope Vicinity of Sta. 15+00	17	4A	2
2.	Typical Riprap on Upstream Slope	16	20A	2
3.	Repaired Section of Riprap, Vicinity of Sta. 50+00	17	15A	3
4.	Example of Eroded Section of Riprap	17	12A	3
5.	Downstream Slope Near Sta. 9+00, Where Seepage Occurs	17	11A	4
6.	Seepage Area on Downstream Slope Near Sta. 9+00	17	10A	4
7.	General View of Downstream Slope of Embankment, Photographed From Near Sta. 4+00	17	9A	5
8.	Outlet of Embankment Drain Pipes, Sta. 10+00	17	7A	5
9.	Outlet Channel From Embankment Drains at Sta. 42+00 and Wet Area Downstream	16	23A	6
10.	Outlet of Embankment Drain Pipes, Sta. 42+00	16	21A	6
11.	Intake Tower and Access Bridge	17	3A	7
12.	Abutment for Access Bridge	17	0A	7
13.	Outlet of 30-inch Water Intake from Segreganset River	16	13A	8
14.	Approach Channel, Photographed From North St.	16	19A	8
15.	South End of Twin 36-inch Concrete Culverts Under North St.	16	16A	9
16.	Emergency Spillway Immediately North of North St., Labor-In-Vain Brook Bed on Left	16	18A	9



NOTE:

PLAN DEVELOPED FROM 1964 PLAN BY WHITMAN & HOWARD, INC. (APPENDIX B-1) AND VISUAL OBSERVATIONS BY HALEY & ALDRICH, INC. ON 19 JULY 1978

LEGEND:

(5) PHOTOGRAPH NUMBER AND DIRECTION OF VIEW

HALEY & ALDRICH, INC.

SOMERSET RESERVOIR DAM
SOMERSET, MA

SITE SKETCH PLAN

SCALE 1"=400'   SEPTEMBER 1978

4160 B49

1. Crest of dam and upstream slope, vicinity of Sta. 15+00

2. Typical riprap on upstream slope

3. Repaired section of riprap, vicinity of Sta. 50+00

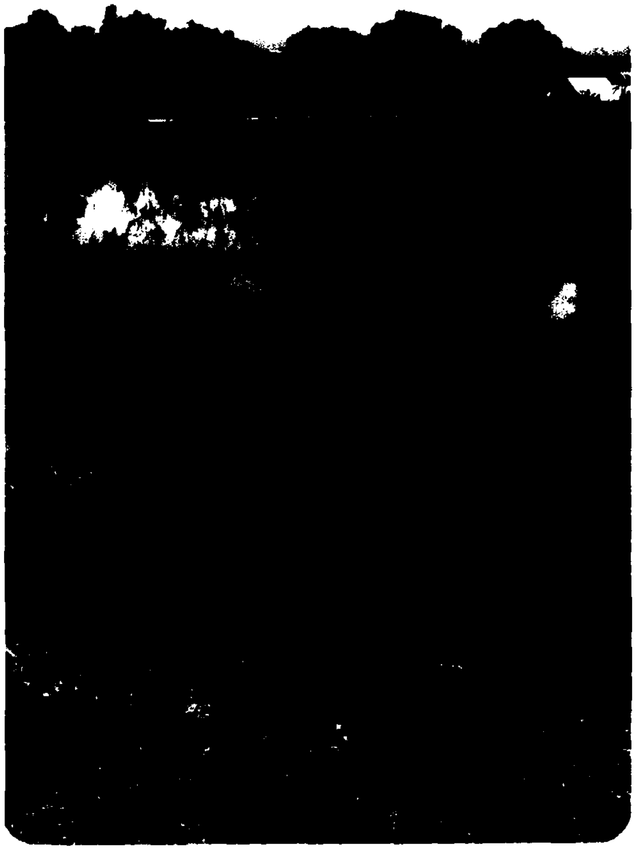
4. Example of eroded section of riprap

5. Downstream slope near Sta. 9+00, where seepage occurs

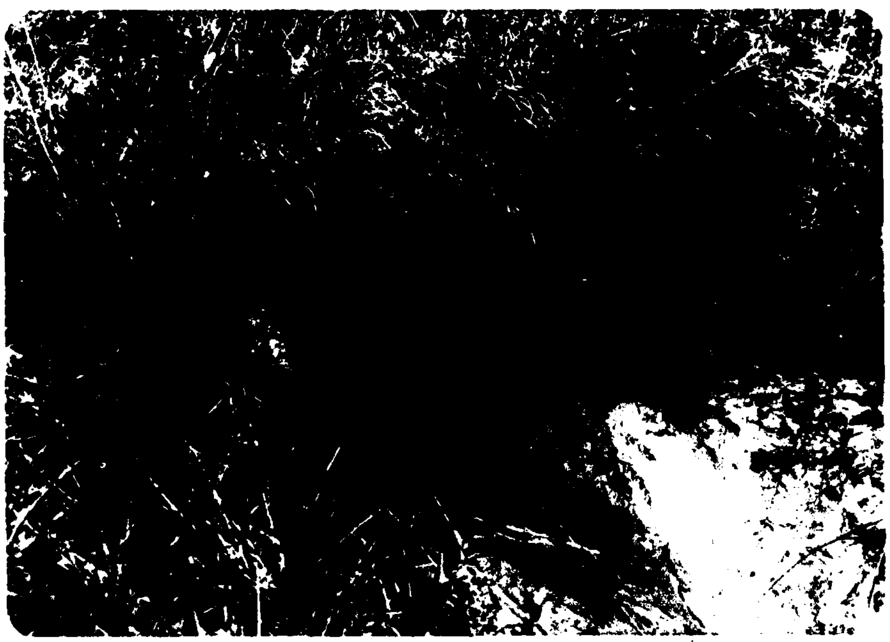
6. Seepage area on downstream slope near Sta. 9+00

7. General view of downstream slope of embankment, photographed from near Sta. 4+00

8. Outlet of embankment drain pipes, Sta. 10+00

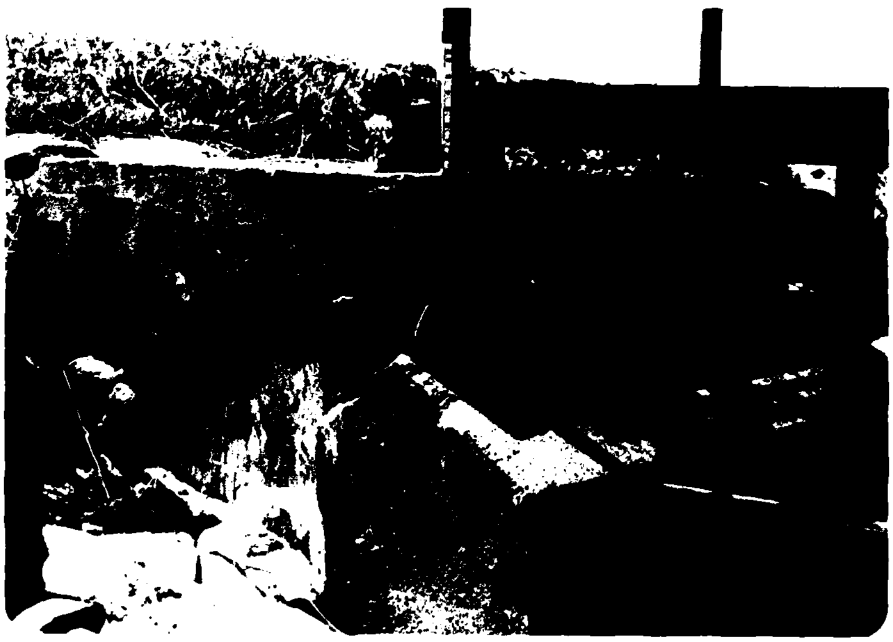


9. Outlet Channel From Embankment Drains at Sta. 42+00 and Wet Area Downstream

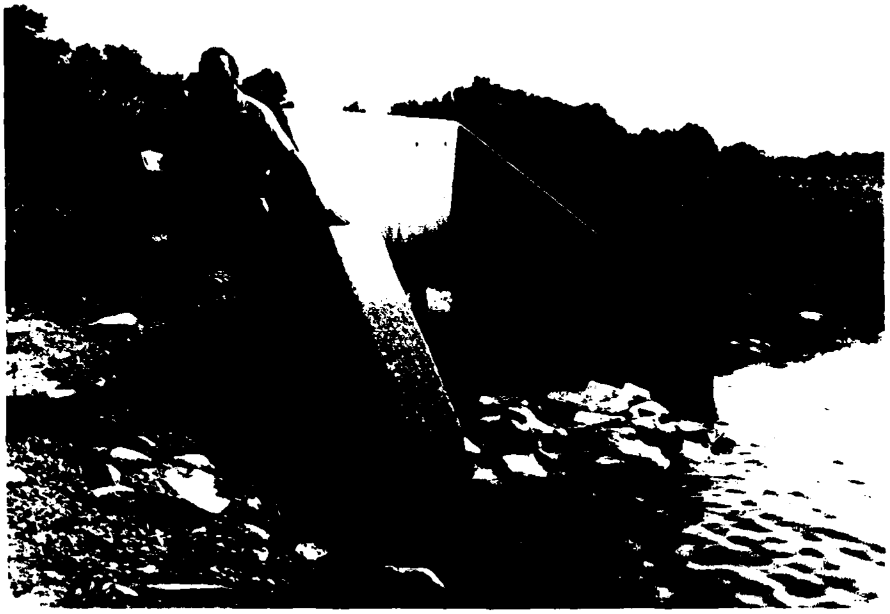


10. Outlet of Embankment Drain Pipes, Sta. 42+00

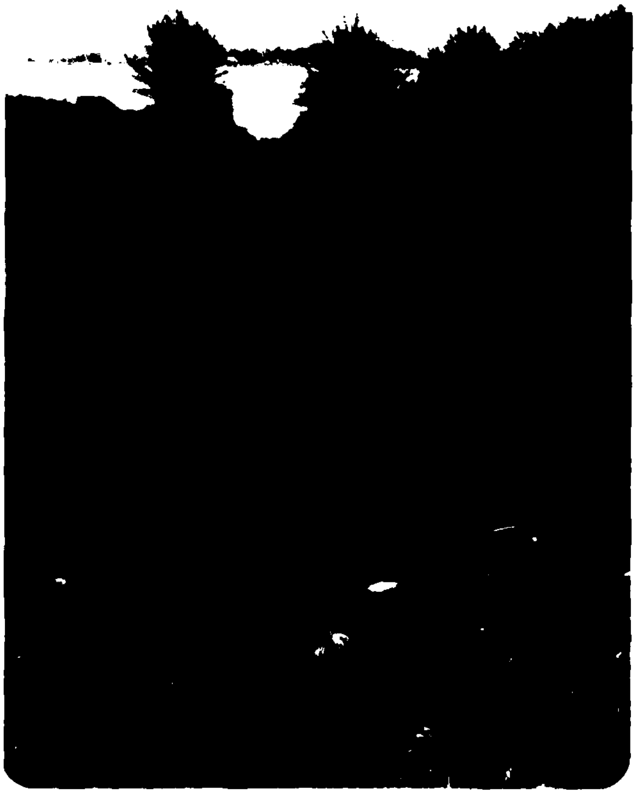
11. Intake tower and access bridge



12. Abutment for access bridge



13. Outlet of 30-inch water intake from Segreganset River



14. Approach channel, photographed from North St.

15. South end of twin 36-inch concrete culverts under North St.

16. Emergency spillway immediately north of North St., Labor-In-Vain Brook bed on left

APPENDIX D
OUTLINE OF DRAINAGE AREA AND HYDRAULIC COMPUTATIONS

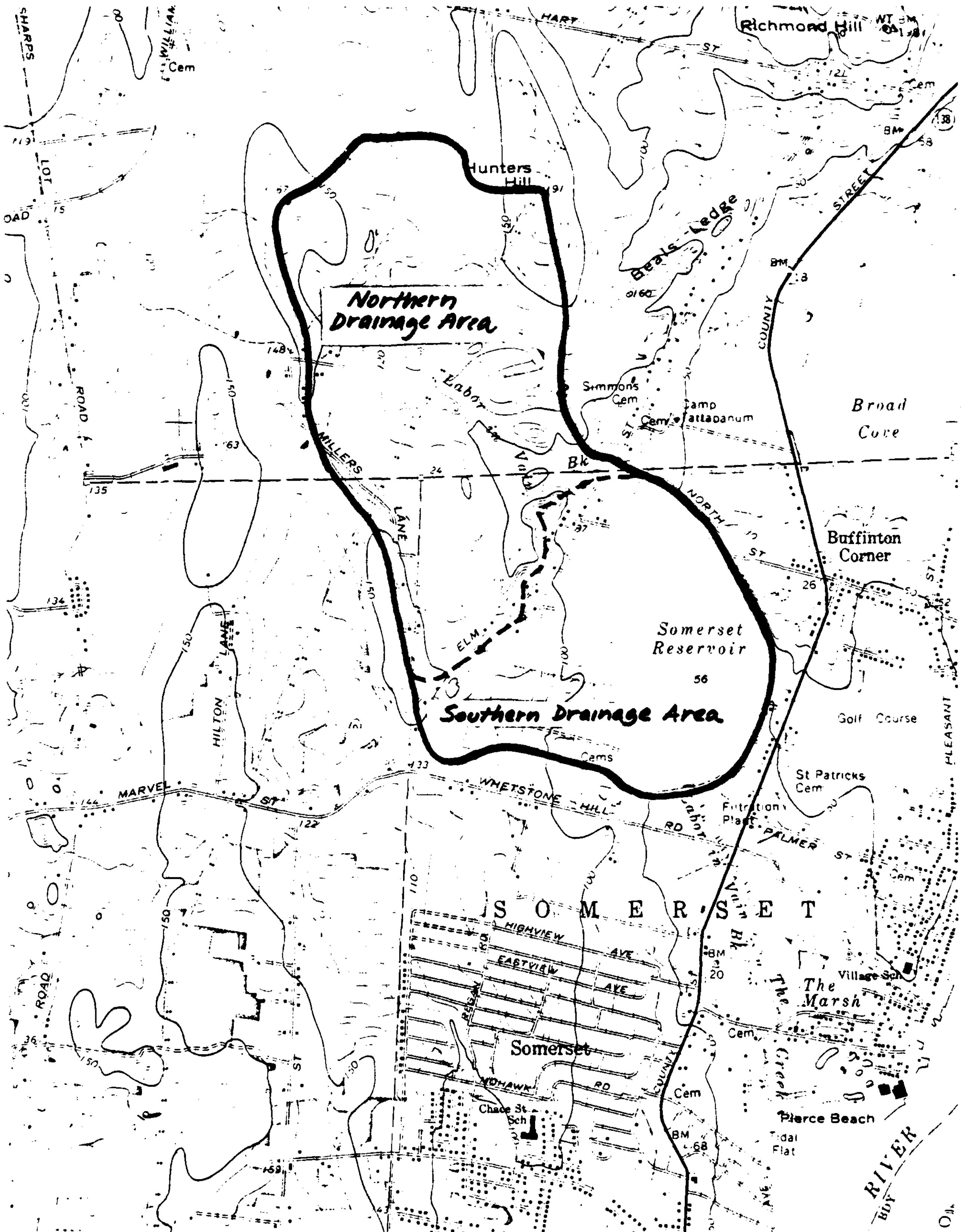
Page No.

OUTLINE OF DRAINAGE AREA

Drainage Area Map — 1

COMPUTATIONS

100-Year Flow and PMF Flow Calculations — 2
Size and Hazard Classification — 11
Stage-Discharge Calculations for Somerset Reservoir — 18
PMF Flow Calculations for Southern Drainage Area — 24
Routing Procedures — 29
Capacity Calculations of Overflow Ditch — 38
Capacity Calculations of Twin 36-in. R.C.P. — 41
Distribution of Northern Drainage Area Flow Between the 36-in. R.C.P. and the Overflow Ditch — 44
Final Reservoir Outflow and Corresponding Water Surface Elevation — 51



CAMP DRESSER & McKEE Inc. Consulting Engineers Boston, Mass.	N	SOMERSET RESERVOIR DRAINAGE AREA SCALE: 1:24,000

CAMP DRESSER & McKEE
Environmental Engineers
Boston, Mass.

CLIENT [illegible] O.I. JOB NO. 561-3-27 PAGE ____
PROJECT Somerset Reservoir DATE CHECKED 8-16-78 DATE 7/31/78
DETAIL Hydrology CHECKED BY [illegible] COMPUTED BY [illegible]

Drainage Area = 922 Acres (1.44 sq. mi.)

Area of Reservoir = 165 Acres (.26 s.m.) @ Elev. 56

Reservoir comprises ~ 18% of Total Drainage Area

Length of Watershed: 3460' overland Flow
4420' Flow in Laoe in Lain Brook

Since D.A. is less than 2000 Acres, use Curve Number Method to determine T_c

$$Lag = \frac{l^{0.8}(S+1)^{.7}}{1900\,Y^{.5}}$$

a. Slope
@ 85% 6700 139'

@ 15% 1130 82'

Y = 1.0326% slope

b. l = 7220'

c. $S = \frac{1000}{CN} - 10$

Find CN - Hydrologic Group C.

	Area	CN	CN x Area
Somerset Reservoir	165	100	16500
Wetland (Swamp)	88	98	8624
Streets, roads	3.5	98	343
Houses (say 1/4 acre)	42	83	3486
Pasture - med.	623.5	77	48010
	922		76963

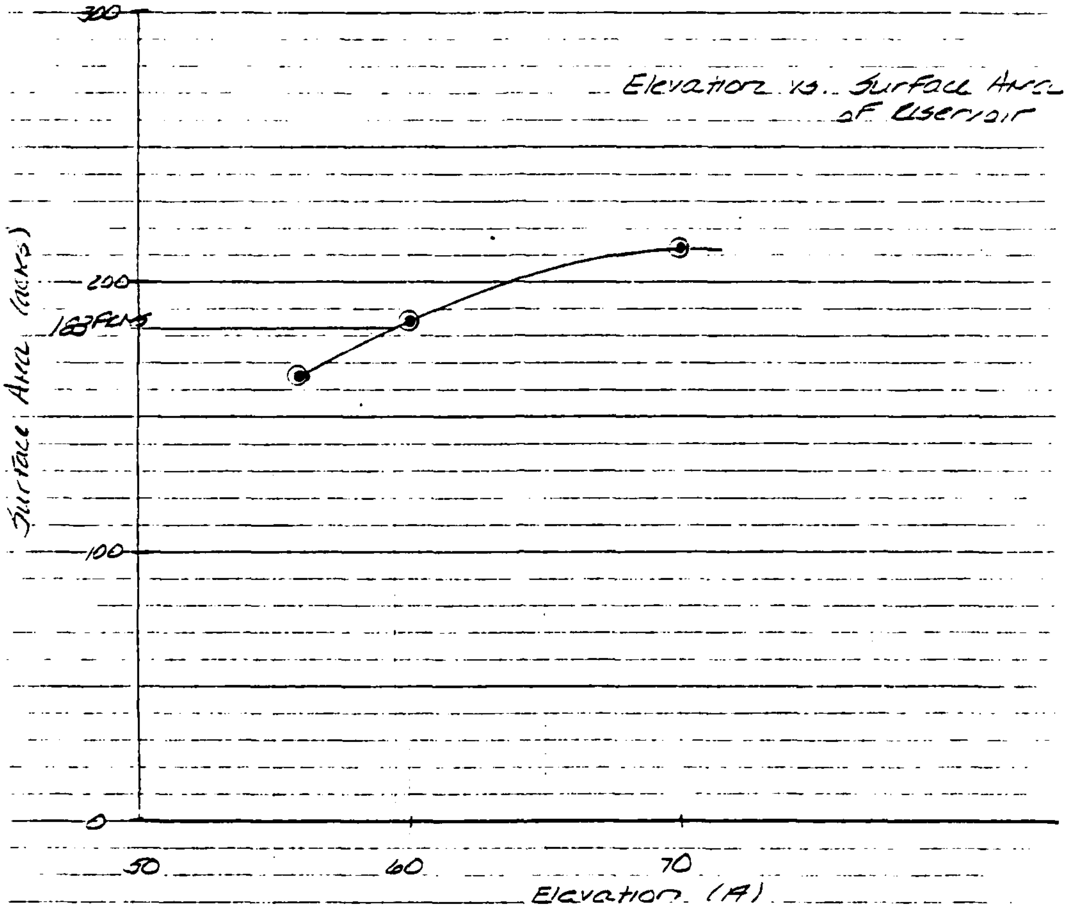
Weighted CN = 83.5 say 84

$S = \frac{1000}{84} - 10 = 1.905$

CAMP DRESSER & McKEE
Environmental Engineers
Boston, Mass.

CLIENT Haley and Aldrich
PROJECT Somerset Reservoir
DETAIL Hydrology
JOB NO. 561-4-2
DATE CHECKED 8-16-78
CHECKED BY [illegible]
PAGE 10
DATE [illegible]
COMPUTED BY [illegible]

Elevation vs. Surface Area of Reservoir



Elev.	Surface Area
56	165 Acres (0.26 s.m.)
60	185 Acres (0.29 s.m.)
70	212 Acres (0.33 s.m.)

Drainage Area = 1.44 sq.mi. = 922 Acres

$$Lag = \frac{7380^{0.8}(2.905)^{.7}}{1900 \times (10326)^{.5}} = 1.43 \text{ hours} \quad (86 \text{ min.})$$

$$T_c = \frac{L}{.6} = 2.39 \text{ hrs} \quad (143 \text{ min})$$

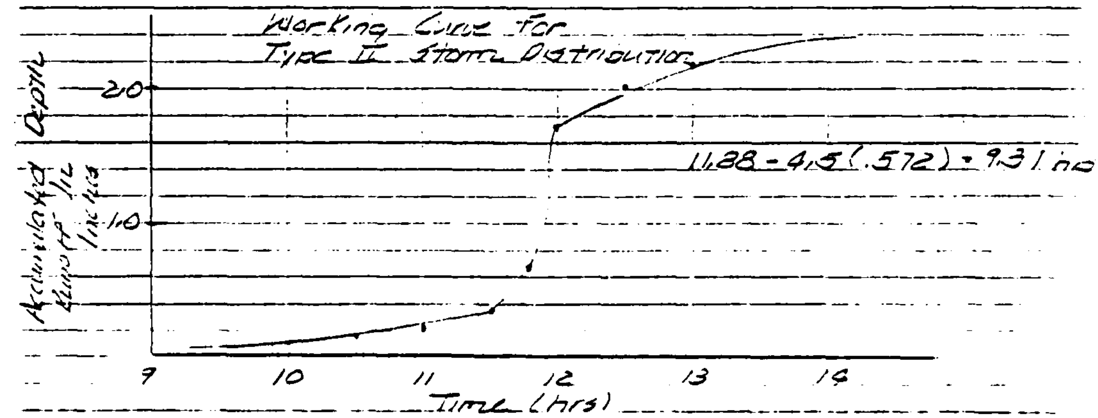
100-year 6-hour Rainfall = 4.9 inches in 6 hours

$$\Delta D = \frac{T_p}{3} = 0.4\,L$$

$$\Delta D = 0.4 \times 1.43 = 0.572 \text{ hours} = (34 \text{ min.})$$

$$7\Delta D = 7 \times 34 \text{ min} = 238 \text{ min} \ (3.97 \text{ hr})$$

Time (hours)	P_x/P_{24}	Mass P (inches)	Mass Q (inches)
10.0	0.181	.89	0.11
10.5	.204	1.00	0.15
11.0	.235	1.15	0.22
11.5	.283	1.39	0.35
11.75	.387	1.90	0.67
12.0	.663	3.25	1.72
12.5	.735	3.60	2.02
13.0	.772	3.78	2.18



Instantaneous Peak Discharges

Increment	Time (hours)	Mass Runoff (inches)	ΔQ	Δq (cfs)	Y	$Y\Delta q$ (cfs)
	9.31	0.06				
ΔQ_1			0.03	12.2	0.2	2.4
	9.88	0.09				
ΔQ_2			0.06	24.4	0.4	9.7
	10.45	0.15				
ΔQ_3			0.07	28.4	0.6	17.1
	11.02	0.22				
ΔQ_4			0.20	81.2	0.8	65
	11.59	0.42				
ΔQ_5			1.38	560.5	1.00	560.5
	12.16	1.80				
ΔQ_6			0.25	101.5	0.67	68.0
	12.73	2.05				
ΔQ_7			0.19	77.2	0.33	25.5
	13.30	2.24				
						Σ 748.2 cfs

say 748 cfs

$$\Delta q_p = \frac{484\ A}{\frac{\Delta D}{2} + L}(\Delta Q)$$

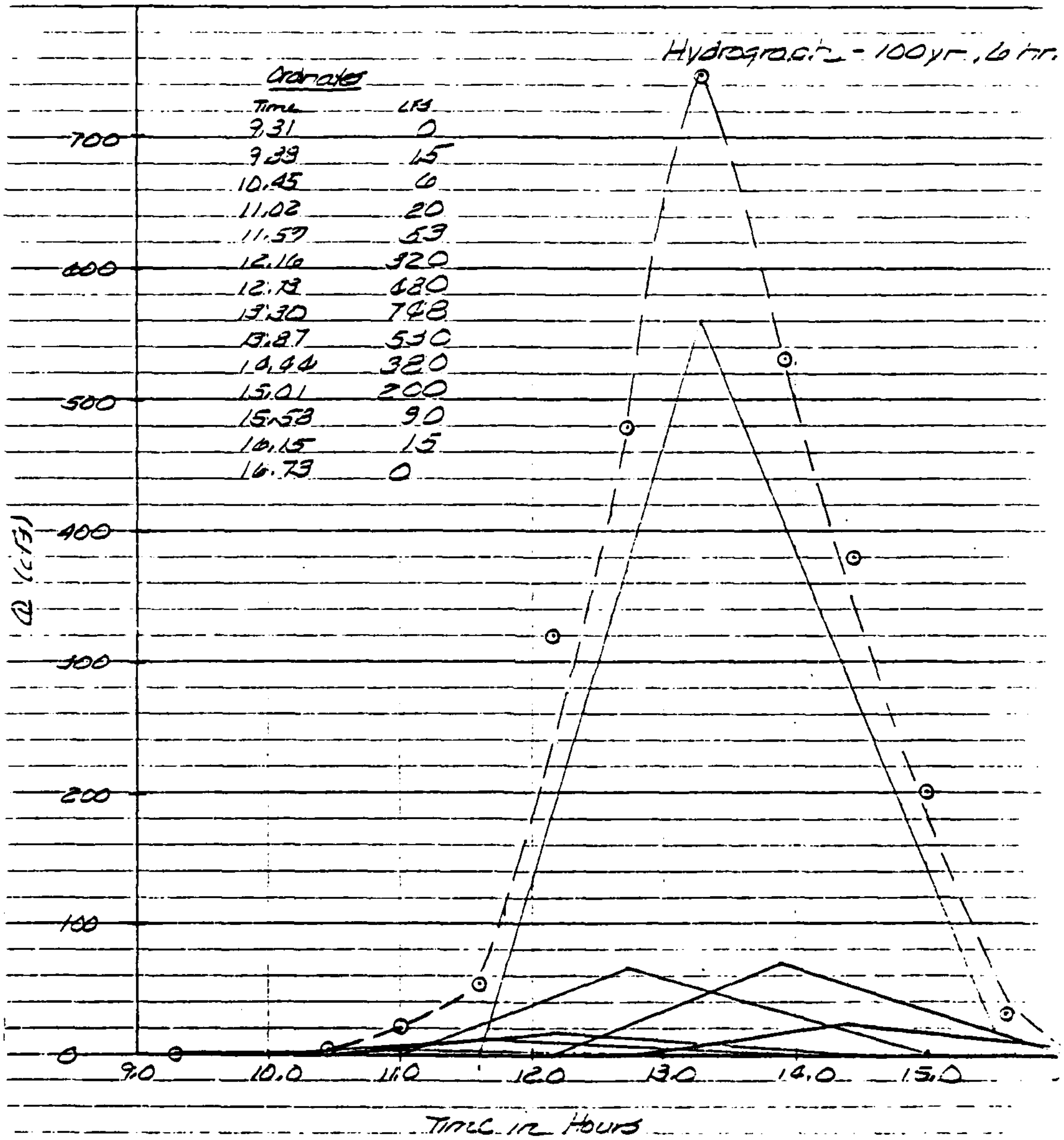
A = 1.44 sq.mi

$$\frac{\Delta D}{2} + L = \frac{.572}{2} + 1.43 = 1.716$$

$$\Delta q_p = \frac{484 \times 1.44}{1.716}\ \Delta Q = 406.2\ \Delta Q$$

CAMP DRESSER & McKEE
Environmental Engineers
Boston, Mass.

CLIENT: [illegible] JOB NO. [illegible] PAGE: [illegible]
PROJECT: Somerset Reservoir DATE CHECKED: 8-16-78 DATE: [illegible]
DETAIL: Hydrology CHECKED BY: [illegible] COMPUTED BY: [illegible]



100-year, 6 hour peak ≅ 748 cfs

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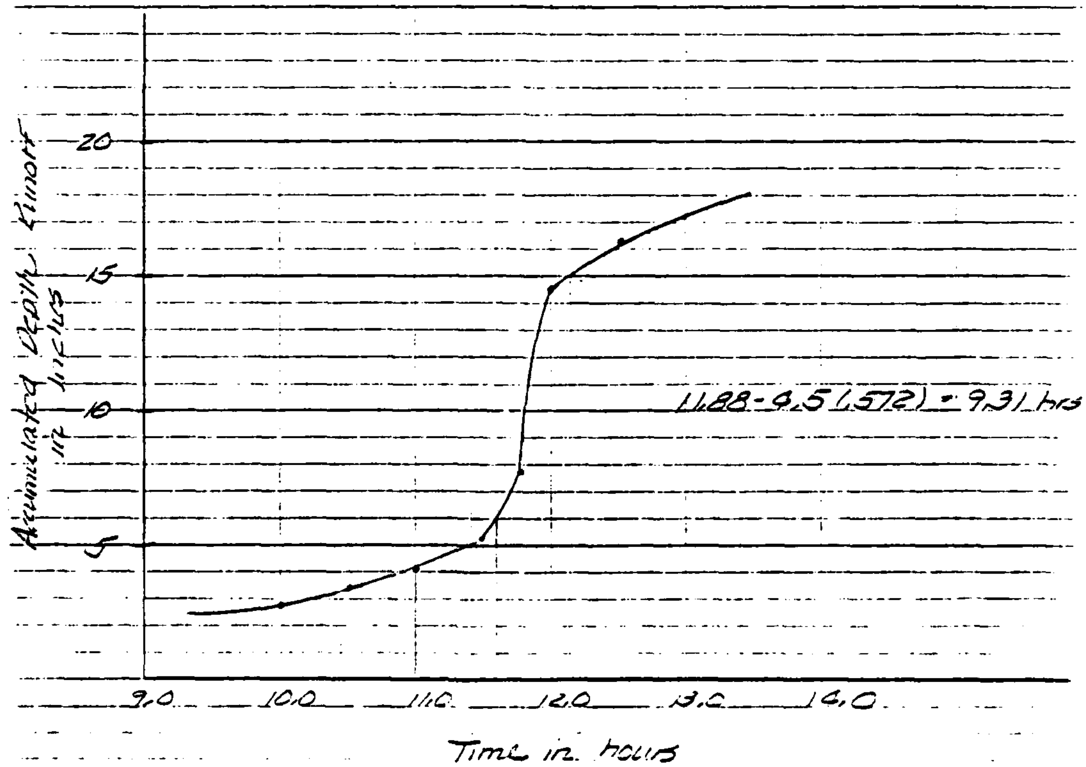
Probable Maximum 6-hr Precipitation - 25 inches in 6 hours

Time (hours)	Px/P6	Mass P (inches)	Mass Q (inches)
10.0	0.181	4.53	2.34
10.5	0.204	5.10	3.36
11.0	0.235	5.83	4.08
11.5	0.283	7.08	5.22
11.75	0.387	9.68	7.72
12.0	0.663	16.58	14.50
12.5	0.735	18.38	16.28
13.0	0.772	19.30	17.19



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PROJECT [illegible] Reservoir DATE CHECKED 8-16-79 DATE 7-31-79
DETAIL Hydrology CHECKED BY [illegible] COMPUTED BY [illegible]

Increment	Time (hours)	Mass Runoff (inches)	ΔQ	Δq (cfs)	Y	$Y\Delta q$ (cfs)
	9.31	2.5				
ΔQ_1			0.2	81	0.2	16
	9.88	2.7				
ΔQ_2			0.6	244	0.4	97
	10.45	3.3				
ΔQ_3			0.8	325	0.6	195
	11.02	4.1				
ΔQ_4			1.9	772	0.8	617
	11.59	6.0				
ΔQ_5			9.0	3656	1.0	3656
	12.16	15.0				
ΔQ_6			1.6	650	.667	433
	12.73	16.6				
ΔQ_7			1.1	447	.333	149
	13.30	17.7				
						Σ 5163

$$\Delta q = \frac{484 A}{\frac{\Delta D}{2} + L}\,\Delta Q = 406.2\,\Delta Q$$

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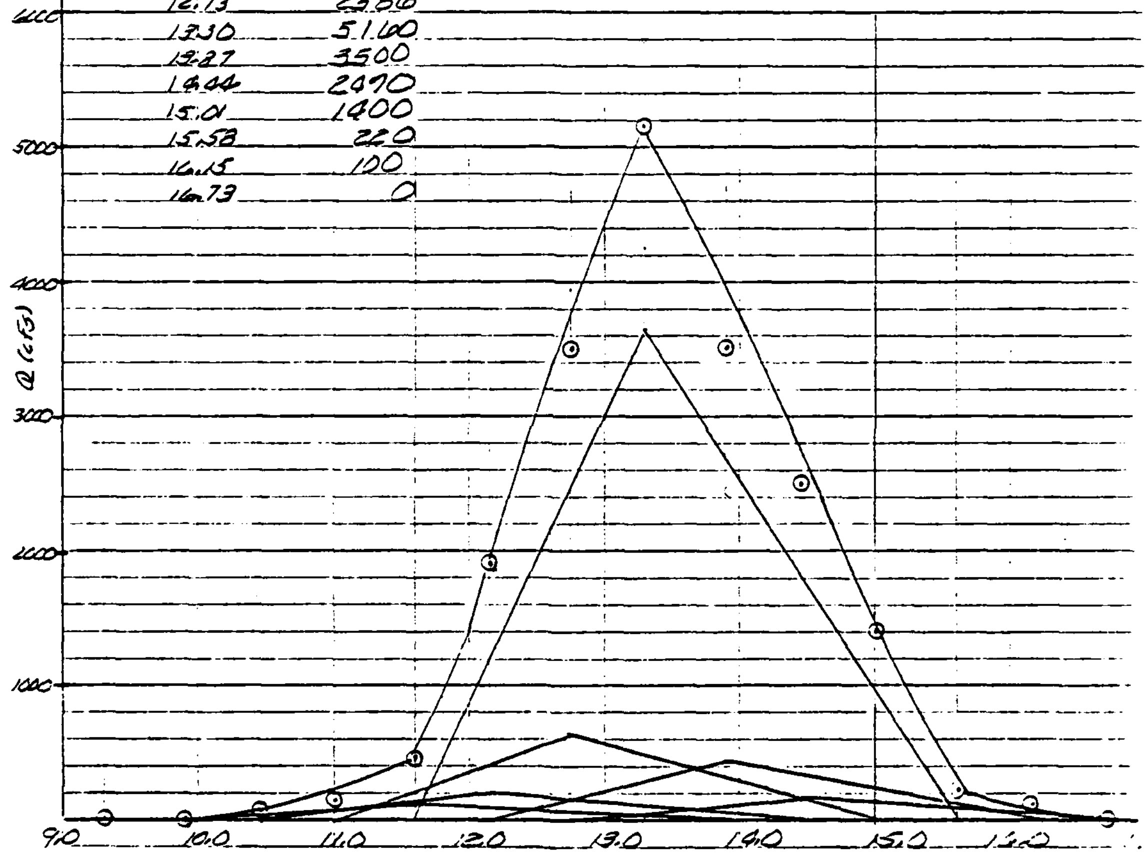
Ordinates

TIME	CFS
9.31	0
9.88	10
10.45	50
11.02	160
11.59	480
12.16	1900
12.73	2536
13.30	5160
13.87	3500
14.44	2470
15.01	1400
15.58	220
16.15	100
16.73	0

Hydrograph
PMF: 25 inches in 6 hours



Time in hours

Peak Flow = 5160 cFs

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Comparison of SCS-TP-149 Flow Values with The COE Maximum Probable Flood, Peak Flow Rates Graph

According to COE Maximum Probable Flood, Peak Flow Rates Graph:

Using Flat & Coastal Curve,

$Q = 930 \text{ cfs}/\text{sq.mi} = 1340$ for 1.44 sq.mi

Try 3/4 way from Flat & Coastal to Rolling
$Q = 1830 \text{ csm} \times 1.44 = 2635 < 5160$ cfs

However, this curve should not be used for such small watersheds (<10 sq.mi.)

This value of 1340 cfs is much different from the value of 5160 cfs obtained from SCS TP-149. I feel that the 5160 cfs value is valid.

$$\frac{Q_1}{Q_2} = \frac{A_1\left(\frac{.894}{A_1^{.048}}-1\right)}{A_2\left(\frac{.894}{A_2^{.048}}-1\right)} = \frac{2\left(\frac{.894}{2^{.048}}-1\right)}{1.44\left(\frac{.894}{1.44^{.048}}-1\right)} = \frac{.9105089}{.9566590}$$

$$\frac{930}{Q_2} = .9517591, \quad Q_2 = 977 \text{ cfs}/\text{sq.mile}$$

$Q_2 \doteq 1407$ cfs (close to 1340 cfs)

O.K. ✓

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Classification of Dam

Size: Height = 59.5' - 18' = 41.5'

Storage (@ top of dam)

@ Elev. 56.0: $\frac{1}{3} \times 165 \text{ Acres} \times 38 \text{ ft} = 2090$ Acre-ft

@ Elev. 59.5: $\frac{183+165}{2} \times 3.5 + 2090 = 2699$ Acre-ft

∴ 41.5' > 40'
2699 Acre-ft > 1000 Acre-ft

so Size Category is Intermediate

Hazard Potential

Estimating Downstream Dam Failure Hydrographs

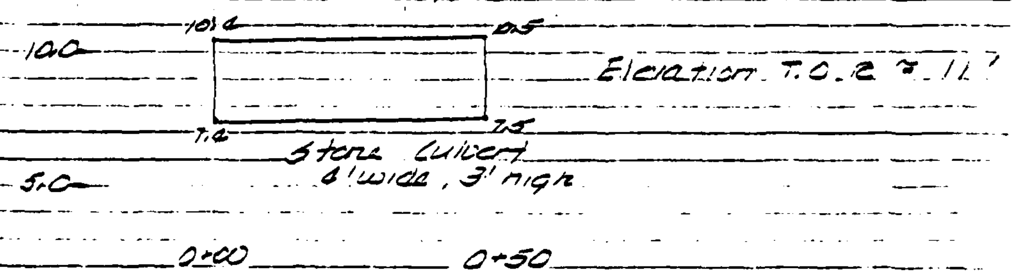
1. S = 2699 Acre-ft @ time of failure
2. $Q_{p_1} = \frac{8}{27} W_b \sqrt{g}\, Y_o^{3/2}$

$W_b = 0.20 \times 6505 \text{ ft} = 1301 \text{ ft}$ (20% because of dam's extraordinary set up compared to usual dam)

$Y_o = 59.5' - 18' = 41.5'$

$Q_{p_1} = \frac{8}{27} \times 1301 \times \sqrt{32.2} \times 41.5^{3/2} = 584{,}795$ cfs

Reach No. 1 - Whetstone Hill Road



Being liberal, assuming no ponding on downstream side:

$$Q_{Full} = \frac{1.49}{.025} R^{2/3} S^{1/2} A$$

$A = 12\ ft^2$
$S = .0020$ (Assume $S_e = S_g$)
$R = \frac{12}{14} = .857$

$Q_{Full} = 27$ cfs

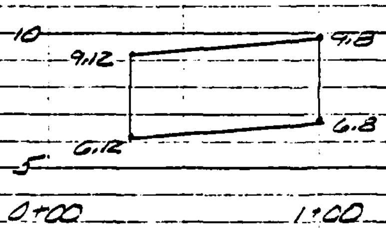
With water at top of road: $Q = 0.77 \times 12 \sqrt{64.4 \times 0.6}$
(Elev. ~11.0)

$Q \approx 57$ cfs

Considering that over 500,000 cfs will come in contact with this culvert, it will most likely be washed away

Try County Street - Route 138

T.O.R. ≈ 11.5'



36" R.C. Pipe

$S = .0097$

$$Q_{Full} = \frac{1.49}{.015} \times \left(\frac{\frac{\pi 3^2}{4}}{\pi 3}\right)^{2/3} \times .0097^{1/2} \times \frac{\pi 3^2}{4}$$

$Q_{Full} = 57$ cfs

With WSEL = 11.5', $Q = 0.79 \times \frac{9\pi}{4} \sqrt{64.4 \times 2.33}$
Top of Road

$Q \approx 69$ cfs

not adequate

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Try Riverside Avenue

Good slope, say 5%

96' CM Pipe

$$Q = \frac{1.49}{.025} R^{2/3} S^{1/2} A$$

$$Q_{full} = \frac{1.49}{.025} \left(\frac{\frac{\pi (8)^2}{4}}{\pi 8} \right)^{2/3} (.0500)^{1/2} \times 16\pi = 1063 \text{ cfs}$$

not adequate

Try South Street

Twin 60" C.M. Pipes

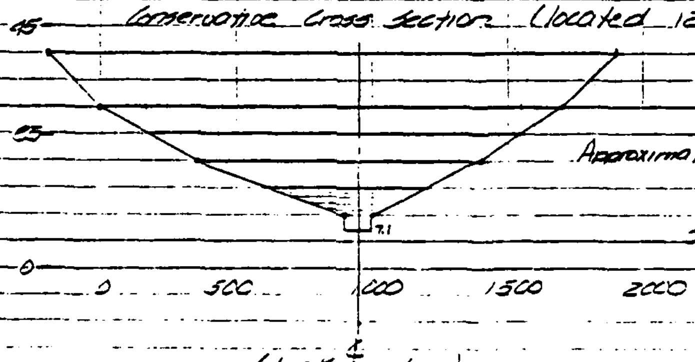
$$Q = \left[\frac{1.49}{.025} \left(\frac{5}{4} \right)^{2/3} (.0500)^{1/2} \times \frac{\pi (5)^2}{4} \right] 2$$

Q_{full} = 607 cfs — not adequate

None of culverts could handle the maximum breach flow. They don't even come near handling the flow under the best conditions (no ponding).

So, assume no controls on stream, take representative cross section, and Finish guidance for estimating downstream dam failure hydrographs method

Conservative Cross Section (located 1220' d.s. from dam)



Approximate Invert = 7.1

n = .030

Slope = $\frac{20-12}{750}$ = 0.0253

(looking d.s.)

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Stage - Discharge Curve Computations for Typical Cross - Section

$$Q = \frac{1.486}{n} R^{2/3} S^{1/2} A$$

Head	Elev.	A	WP	R	S	Q
0	7.1	0	0	0	.0105263	0
0.9	8.0	90	101.8	0.884		421
1.9	9.0	190	103.8	1.830		1445
2.9	10.0	290	105.8	2.741		2888
3.9	11.0	440	205.8	2.138		3712
4.9	12.0	705	320.8	2.197		6058
7.9	15.0	2040	605.9	3.367		23278
12.9	20.0	6040	1056	5.720		93229
17.9	25.0	11615	1376	8.441		244886
22.9	30.0	18840	1696	11.108		477055
32.9	40.0	36890	2077	17.572		1,269,305

From Graphs on Pages 13 & 14, @ Q = 584795 cfs,

Elevation = 31.5 ft (24.4 ft. of head)

Area = 20,020 ft² = 0.4596 Acres

Reach ≈ 1400' long (to County Street)

A. V_1 = 1400 x .4596 = 643.4 Acre-ft < ½ S (1350 Acre-ft)

B. Q_{P2} (TRIAL) = 584795 $\left(1 - \frac{643.4}{2679}\right)$ = 445389 cfs ≡ 31.29.0

C. A_2 = 16350 ft² = .3753 Acre

V_2 = 1400' x .3753 Acre ≈ 525 Acre-ft.

D. $\frac{V_1 + V_2}{2} = \frac{643 + 525}{2}$ ≈ 584 Acre-ft

Q_{P2} = 584795 $\left(1 - \frac{584}{2679}\right)$ ≈ 458240 cfs

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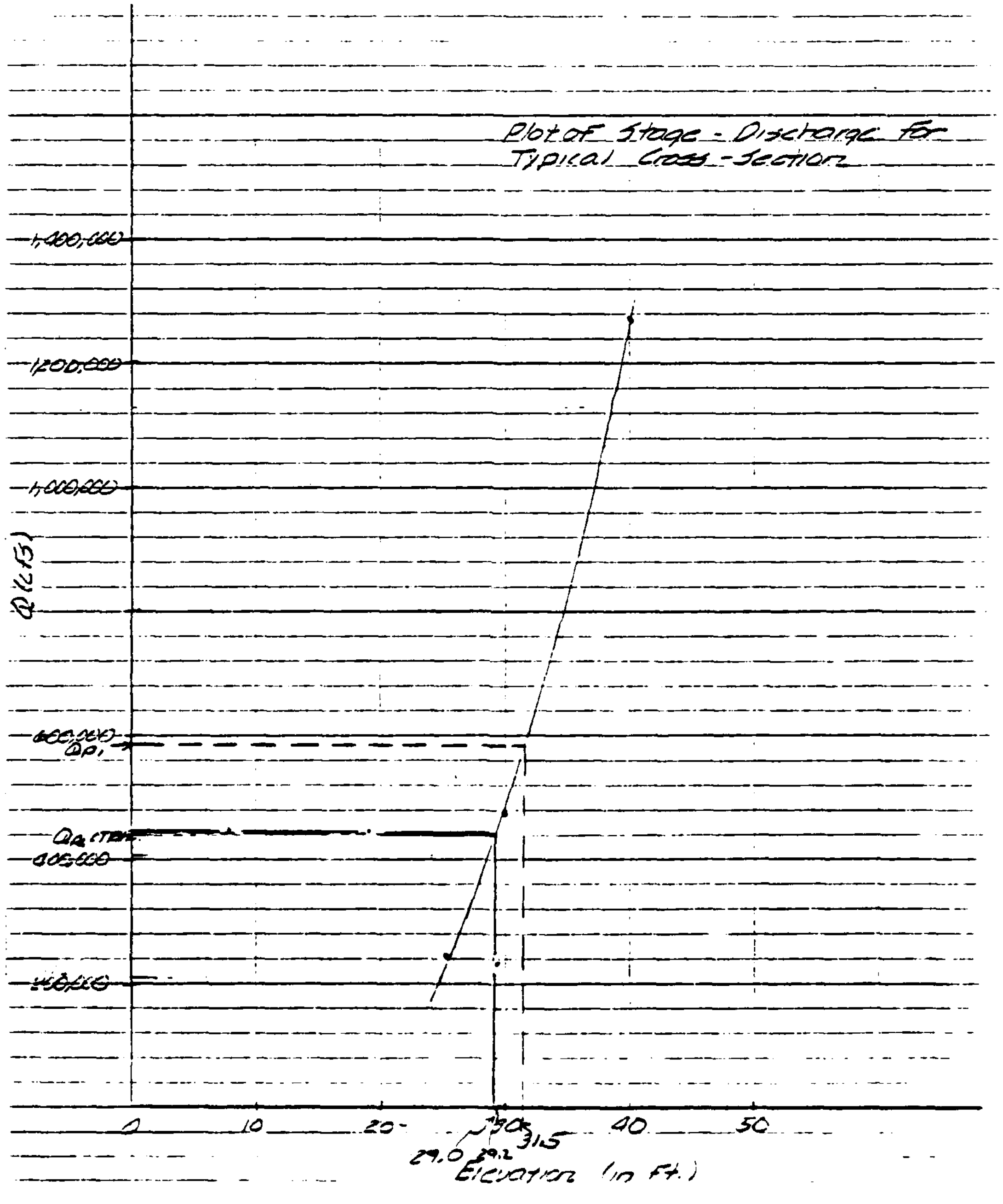

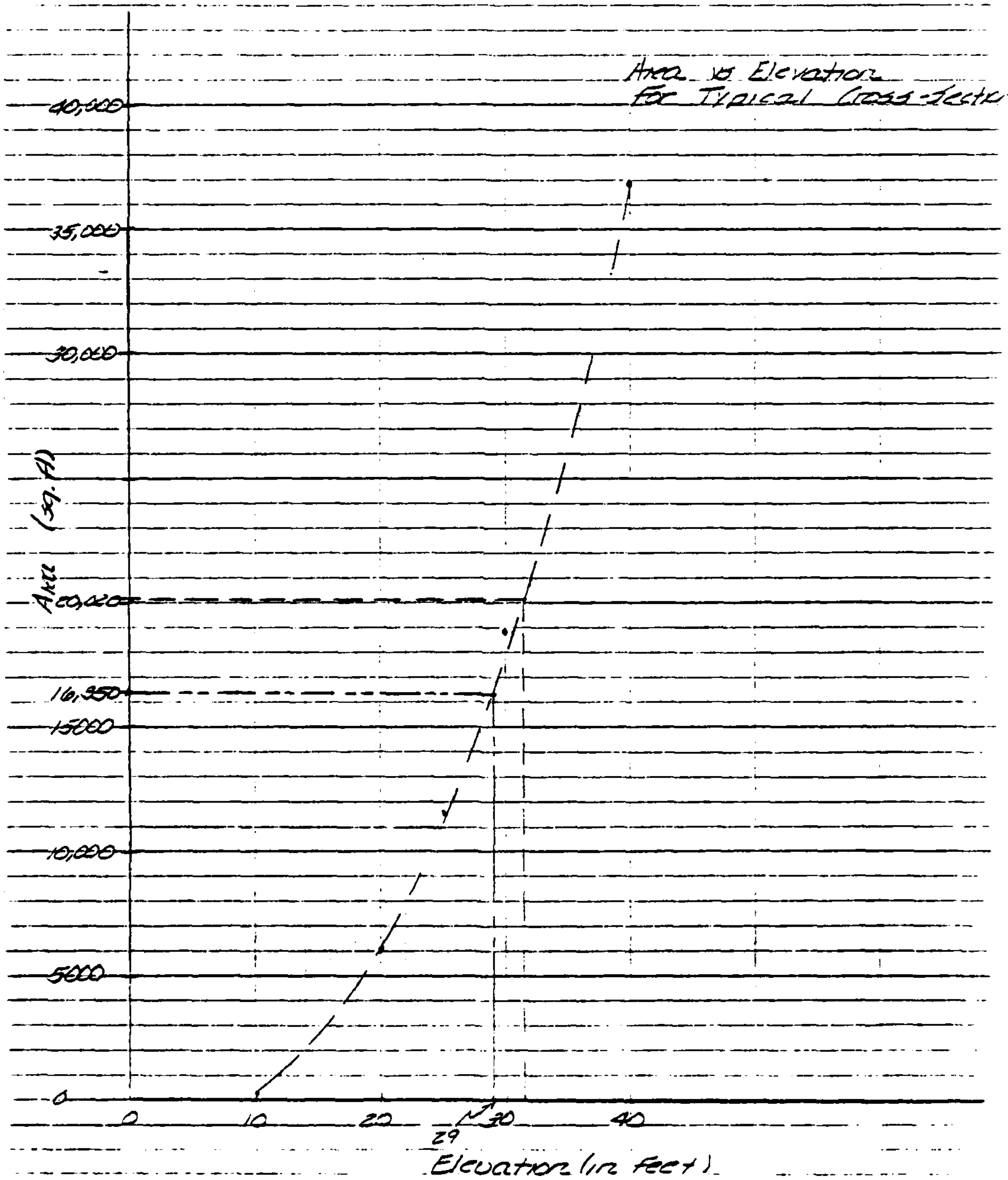
Area vs Elevation
for Typical Cross-Section
Area (sq. ft)
40,000
35,000
30,000
20,000
16,350
15000
10,000
5000
0
0
10
20
29
30
40
Elevation (in feet)

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— • — At Q_{P2} = 458,260 cfs Elevation = 29.2 ft at Typical Cross Section (22.1' of head)

This amount of head will inundate approximately 10 homes, a shopping center and a Filtration Plant within this Reach. Therefore, the Hazard Potential Classification Category is High.

Downstream of this reach, the area becomes even flatter, in contour and slope. However, [illegible] structures are located at a lower elevation than in the upper reach. So, without doing these calculations it also appears that only a few of inhabitable structures would be affected.

So, according to the COE parameters, the Spillway Design Flood (SDF) is the PMF.

The PMF = 5100 cfs (unrouted)

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Approximate Invert of 20" Class 250 M.J. Ductile Iron Pipe = 18.5' (upstream); 14.5' downstream

Slope of Pipe = 0.02
Top of Dam = 59.5 (crown of roadway)
Length of Pipe = 200 ft
Downstream Invert of 20" = 14.5'

Hazen Williams

$$h_L = f \frac{L V^2}{D 2g}$$

$f = .0231$
$L = 200$ ft
$D = 20" = 1.667$ ft
$g = 32.2$ '/s²

$$V = \frac{1.49}{.013} R^{2/3} S^{1/2}, \quad V = \frac{1.49}{.013}\left(\frac{20}{12 \times 4}\right)^{.667}(.02)^{1/2}$$

$$V = 9.04 \text{ '/s}$$

$$h_L = .0231\left(\frac{200' \times (9.04 \text{ '/s})^2}{1.667' \times 64.4 \text{ '/s}^2}\right) = 3.52 \text{ ft}/200 \text{ ft}$$

so $h_L = 17.6$ ft/1000 ft

From Hazen Williams formula, (C = 125), Pipe is 14 years old but never used. (cement lined)

$$Q = 0.279\, C\, D^{2.63} S^{0.54}$$

$$Q = 0.279 \times 125 \times \left(\frac{20}{12}\right)^{2.63} \times \left(\frac{h_L}{1000}\right)^{.54}$$

$Q \doteq 15.1$ cfs

When water surface is at elevation 59.5,

$H = 59.5 - 14.2 = 43.3$ ft (assume no ponding; because of small amount that gets through the 20" D.I. pipe, the stream could handle it.

14.2 = top of pipe

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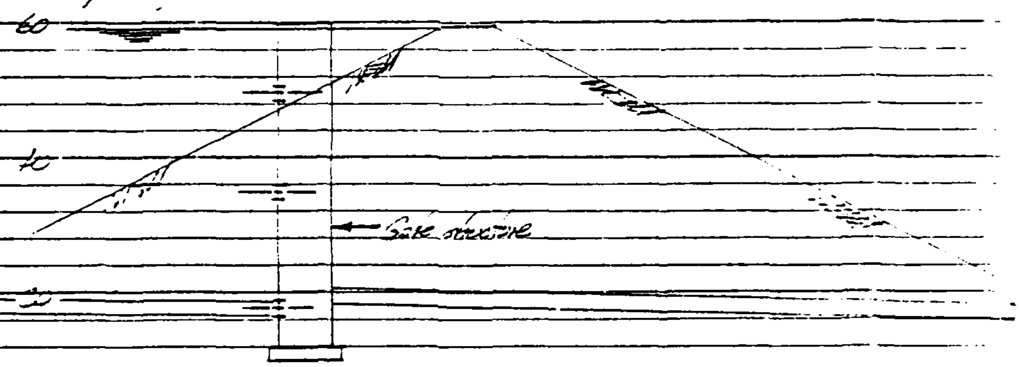
Capacity of 20" C.I. line



Consider inlet loss of 0.5 $\frac{V^2}{2g}$ @ gate structure and outlet loss of 1.0 $\frac{V^2}{2g}$ at d/s end of line — say 1.5 $\frac{V^2}{2g}$ total (0.3? $\frac{V^2}{2g}$ re d/s bend [illegible])

Assume Q = 56 cfs,

then $V_{ave} = \frac{56}{\pi \frac{(20/12)^2}{4}} = 25.67$ fps $\quad \frac{V^2}{2g} = 10.23$ ft

$1.5 \frac{V^2}{2g} = 15.35$ ft $\quad -15.35 \quad 26.55'$

$h_f = \frac{26.55'}{200'} = 0.1322$ for C = 125 Q = 75 cfs too high!

Assume Q = 65 cfs,

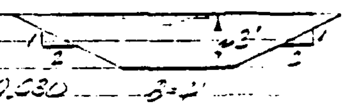
then $V_{ave} = \frac{65}{\pi \frac{(20/12)^2}{4}} =$ [illegible] fps $\quad \frac{V^2}{2g} = 13.13$ ft

$1.5 \frac{V^2}{2g} =$ [illegible] ft $\quad$ 31.3?

$h_f = \frac{21.32'}{200'} = 0.1066$ for C = 125 Q = 63 cfs close enough!

Use Q = 64 cfs

Now check d/s channel conditions



$S = \frac{13.0-11.0}{350} = \frac{2.0}{350} = 0.00571$

$Q = A \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$

$Q = 16.0 \frac{1.486}{0.030} (1.24)^{\frac{2}{3}} (0.00571)^{\frac{1}{2}} = 69.0$ cfs > 64 cfs - so OK.

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Pressure Flow

$Q = CA\sqrt{2gH}$

C = 0.47 for concrete pipe (D.I. is [illegible])

$Q = 0.47 \times \frac{\pi (\frac{20}{12})^2}{4} \sqrt{64.4 \times 43.3}$

Q ≅ 57 cfs ; Pipe is actually too long to be considered an orifice or short tube (See page 17).
So, at WSEL = 57.5, Q pipe = 57 cfs okay

When water begins to overtop dam, the Q value increases quickly

Treat top of dam as broad-crested weir, breadth = 10'.

10' 10'

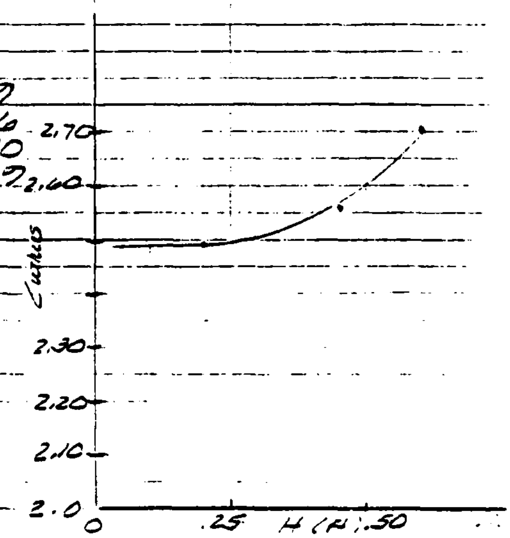
59 —

58 —

C Values:

H	C
0.2	2.47
0.4	2.56
0.6	2.70
0.8	2.67

L = 6505'



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Stage Discharge Relationship for Somerset Dam

Head	Elev.	$Q_{spillway}$	Q_{pipe}	Q_{TOTAL}
0.1	59.6	510	57	567
0.2	59.7	1449	57	1506
0.3	59.8	2633	57	2760
0.4	59.9	4213	57	4270
0.5	60.0	5980	57	6037

Drainage Areas

Northern D.A.1. Upstream of North Street = 560 Acres
Southern D.A.2 Downstream of North Street (including reservoir) : 362 Acres

Northern Drainage Area Flow Computations

Note: Since the SCS TP 147 computations were based on a lag time and slope which was derived from the Labor in Vain Brook Watershed (above Somerset Reservoir), in order to determine the flow from this area, another SCS Method, published in Section 4 Hydrology based on drainage areas will be used.

$$\frac{Q_1}{Q_2} = \frac{A_1^{\left(\frac{.894}{A_1^{.048}} - 1\right)}}{A_2^{\left(\frac{.894}{A_2^{.048}} - 1\right)}}$$

$Q_1 = 5160$ cfs $= 3582$ cfs/s.m.
$A_1 = 922$ Acres
$A_2 = 560$ Acres

$$\frac{3582}{Q_2} = \frac{922^{\left(\frac{.894}{922^{.048}} - 1\right)}}{560^{\left(\frac{.894}{560^{.048}} - 1\right)}}$$

$$\frac{3582}{Q_2} = \frac{.08214}{.116176}$$, $Q_2 = 4721$ cfs/sm

$\boxed{Q_2 = 4131 \text{ cfs}}$

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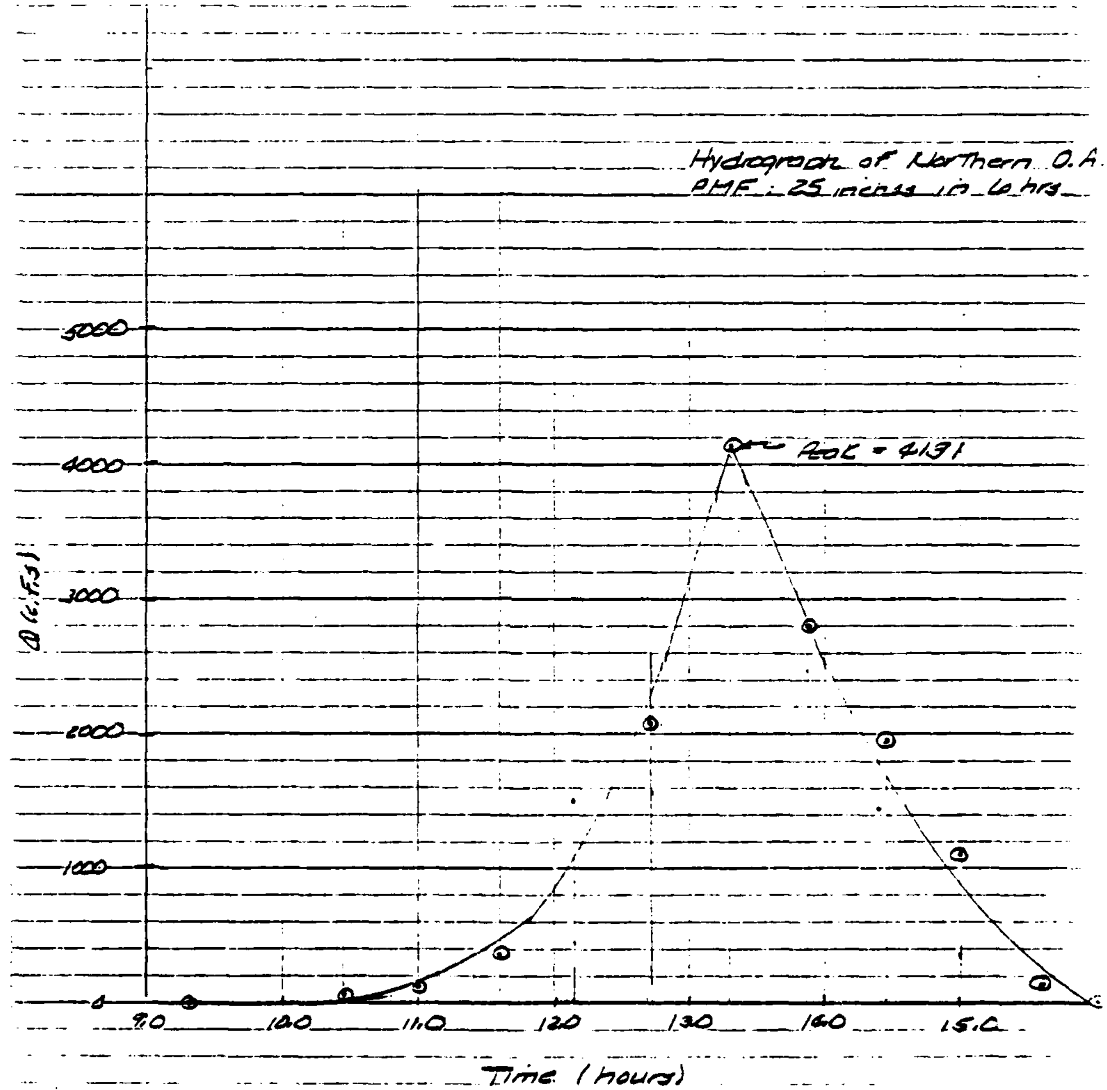

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Stage - Discharge Relationship for Somerset Dam

Head on Spillway (Top of Dam)	Elev.	Q spillway	Q pipe	Q total
—	56	0	54	54
—	57	0	55	55
—	58	0	56	56
—	59	0	56	56
—	59.5	0	57	57
0.1	59.6	510	57	567
0.2	59.7	1449	57	1506
0.3	59.8	2683	57	2740
0.4	59.9	4213	57	4270
0.5	60.0	5980	57	6037

Assume no ponding on downstream side, free flow from outlet of 20" (Top of pipe - 16.2')

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Downstream of North Street Drainage Area Flow Computations

Not including Reservoir

1. Watershed length = 3500'

2. Slope

15%	525	85
85%	2975	150

Slope = .026531, 2.6531 %

3. CN - Hydrologic Soil Group C

	Area	CN	CN x Area
Pasture	187	77	14399
Impervious	10	83	830
	197		15229

Weighted CN = 77.3 say 77

$S = \frac{1000}{77} - 10 = 2.987$

$Lag = \frac{3500^{.8} (3.987)^{.7}}{1900 (2.6531)^{.5}} = 0.582$ hours

$T_c = \frac{L}{.6} = .97$ hrs

Maximum Probable Precipitation: 25 inches in 6 hours

$\Delta D = \frac{T_c}{3} = 0.4L$

$\Delta D = .4 \times .582 = .233$ hours

$7\Delta D = 7 \times .233$ hrs = 1.631 hours

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DETAIL Hydrology

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COMPUTED BY [illegible]

Time (hours)	P/ Ptot	Mass P (inches)	Mass Q (inches)
10.0	.181	4.53	2.84
10.5	.204	5.10	3.36
11.0	.235	5.88	4.08
11.5	.283	7.08	5.22
11.75	.387	9.68	7.72
12.0	.663	16.58	14.50
12.5	.735	18.38	16.28
13.0	.772	19.30	17.19

11.88 - 4.5(.233) = 10.832

See Pg. 5 Graph of Accumulated Depth Runoff

Increment	Time (hrs)	Mass Runoff (inches)	ΔQ	Δq (cfs)	Y	YΔq (cfs)
	10.83	3.9				
ΔQ_1			.3	64	.2	13
	11.06	4.2				
ΔQ_2			.7	149	.4	60
	11.30	4.9				
ΔQ_3			.8	171	.6	103
	11.53	5.7				
ΔQ_4			2.0	427	.8	342
	11.76	7.7				
ΔQ_5			6.8	1451	1.0	1451
	12.00	14.5				
ΔQ_6			.7	149	.667	99
	12.23	15.2				
ΔQ_7			.8	171	.333	57
	12.46	16.0				
						Σ 2125 cfs

$$\Delta q = \frac{484 \times A}{\frac{\Delta D}{2} + L} \Delta Q = \frac{484 \times .308}{\frac{.233}{2} + .582} \Delta Q$$

$$\Delta q = 213.4 \ \Delta Q$$

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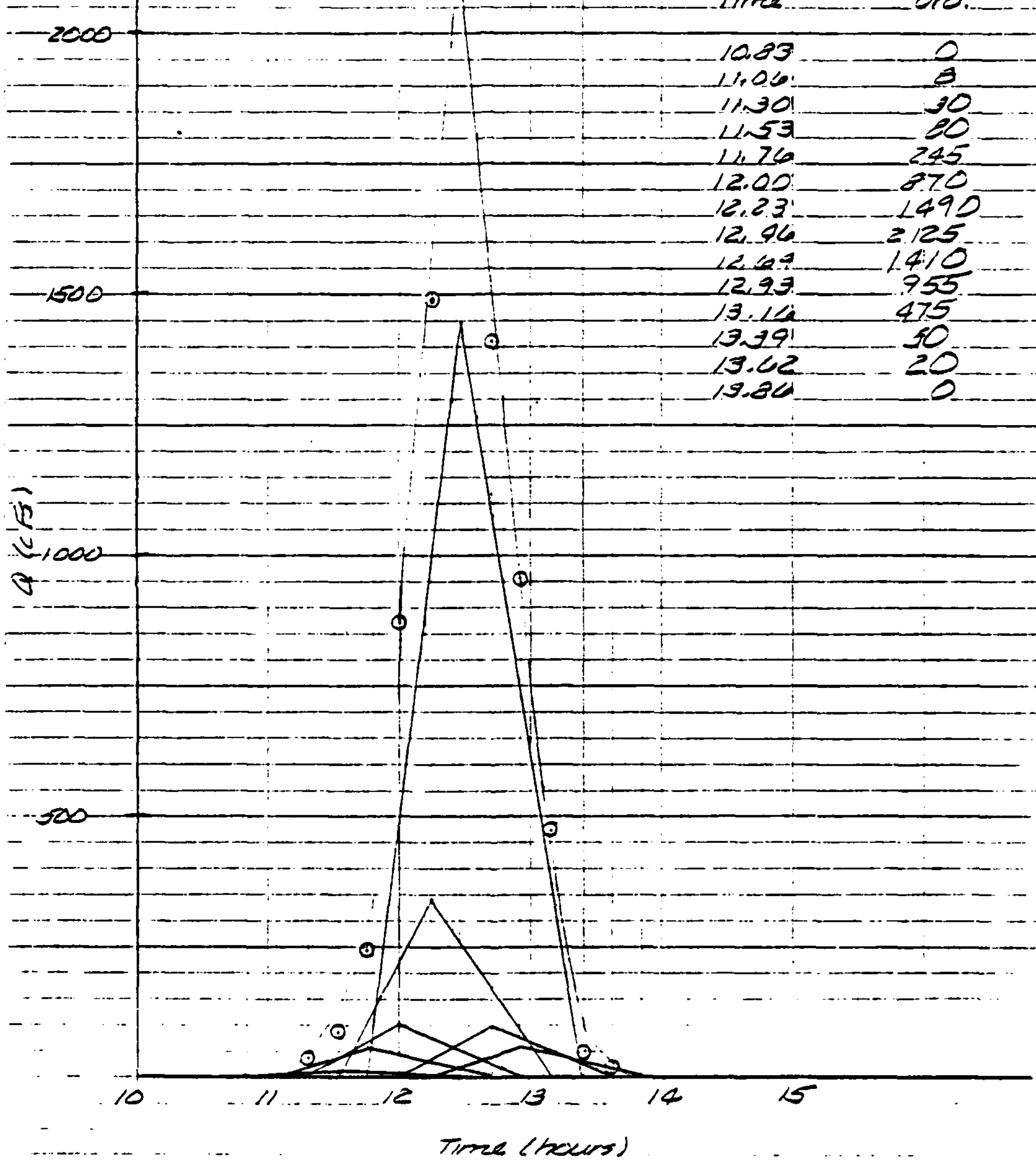
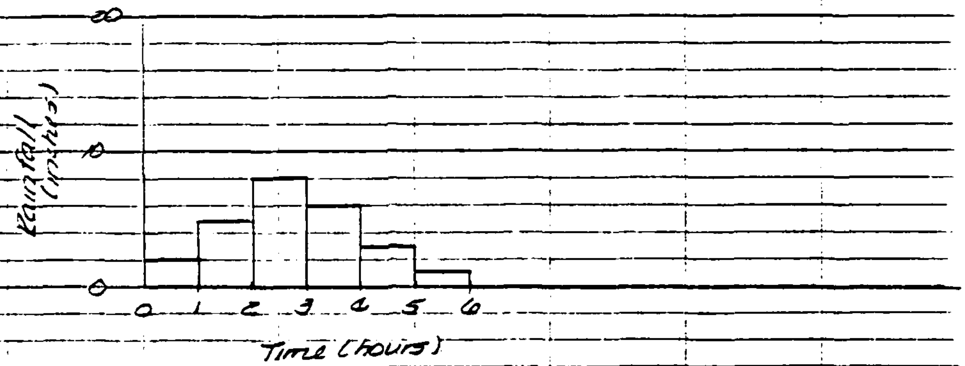
Hydrograph - Southern O.A. Land Contribution

Time	Ordinate
10.83	0
11.06	8
11.30	30
11.53	80
11.76	245
12.00	870
12.23	1490
12.46	2125
12.69	1410
12.93	955
13.16	475
13.39	50
13.62	20
13.86	0

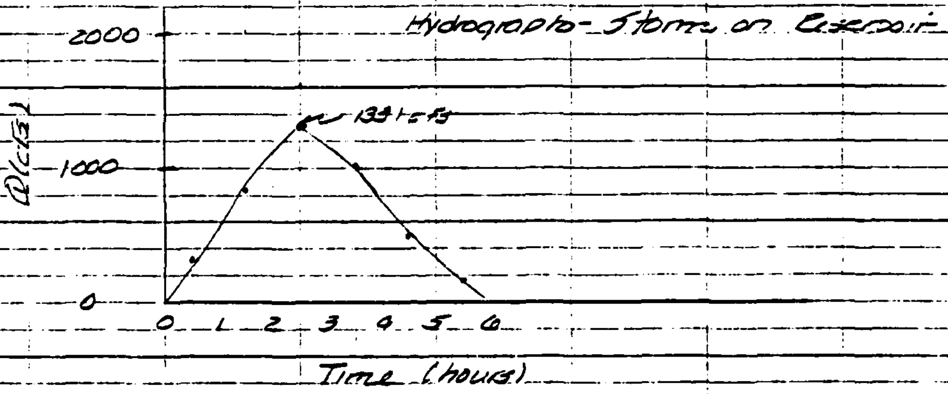


*Flow Computations - Rainfall Directly on Somerset Reservoir

25 inches in 6 hours



Area of Reservoir (@ elev. 56) = 165 Acres



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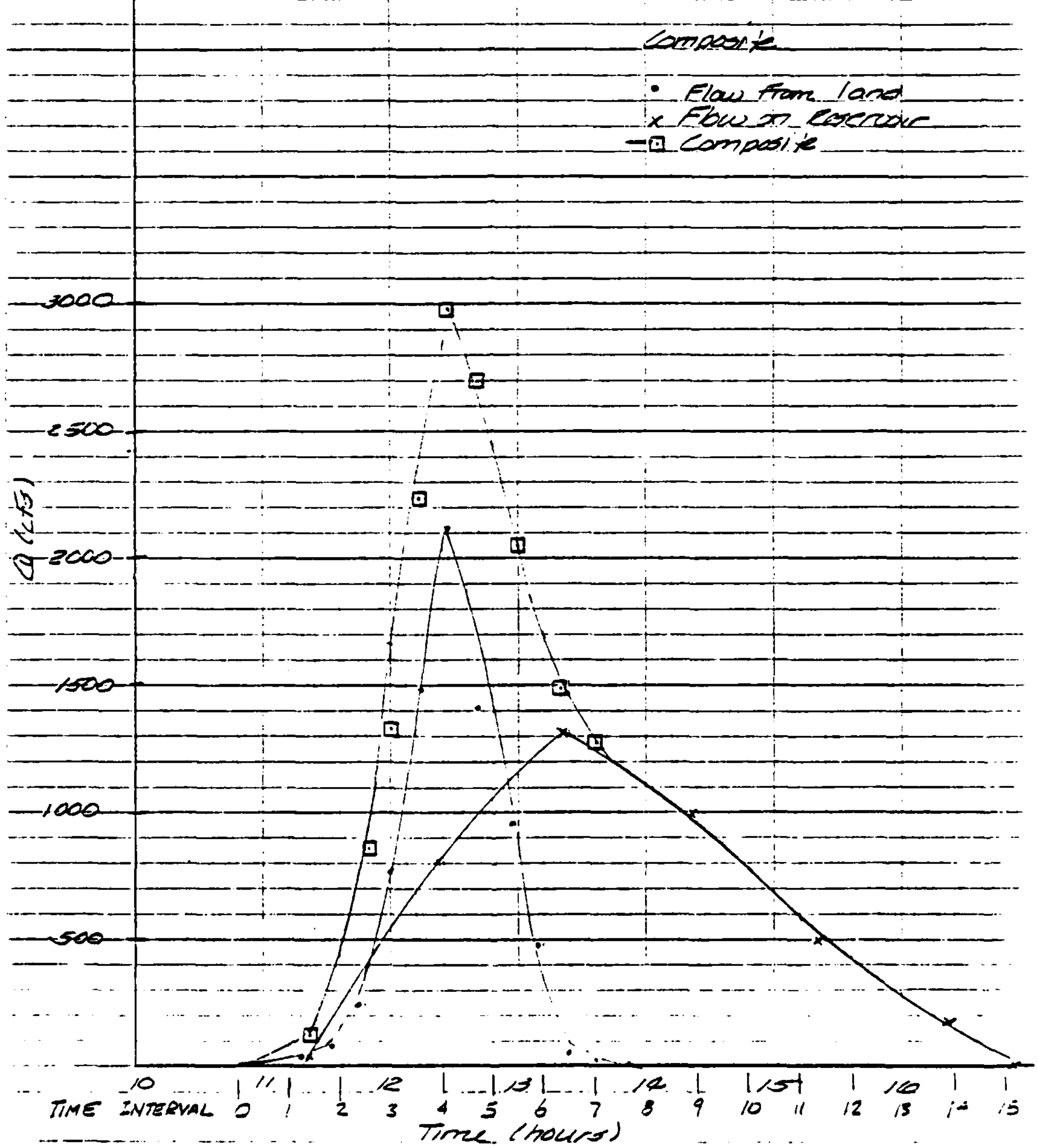
Southern Drainage Area Inflow Hydrograph



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Head on Spillway H, ft.	Reservoir Area A, acres	Calculated Outflow Q, cfs	Calculated Storage S above Spillway level acre-ft	Funtional Rates of Storage: $\frac{S}{\Delta t}$ cfs	$\left(\frac{S}{\Delta t} - \frac{Q}{2}\right)$ cfs	$\left(\frac{S}{\Delta t} + \frac{Q}{2}\right)$ cfs
0	182.7	0	0	0	0	0
0.1	183.2	567	18.3	553	269	836
0.2	183.6	1506	36.6	1108	355	1861
0.3	184.1	2740	55	1664	294	3034
0.4	184.6	4270	73.5	2222	86	4356
0.5	185.0	6037	91.9	2781	-238	5799

Δt = 24 min. = 1440 sec.

* Superseded by Page 32.

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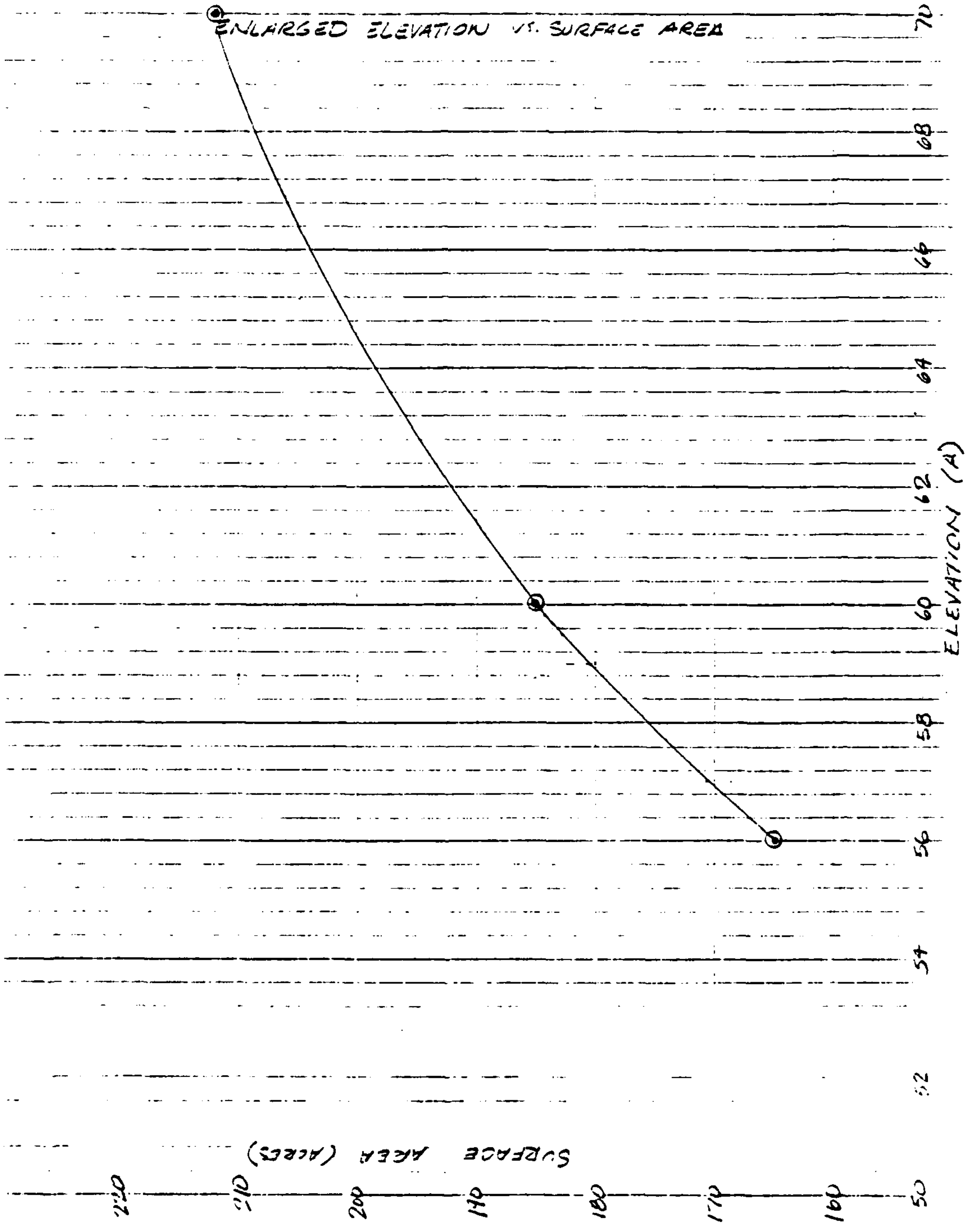

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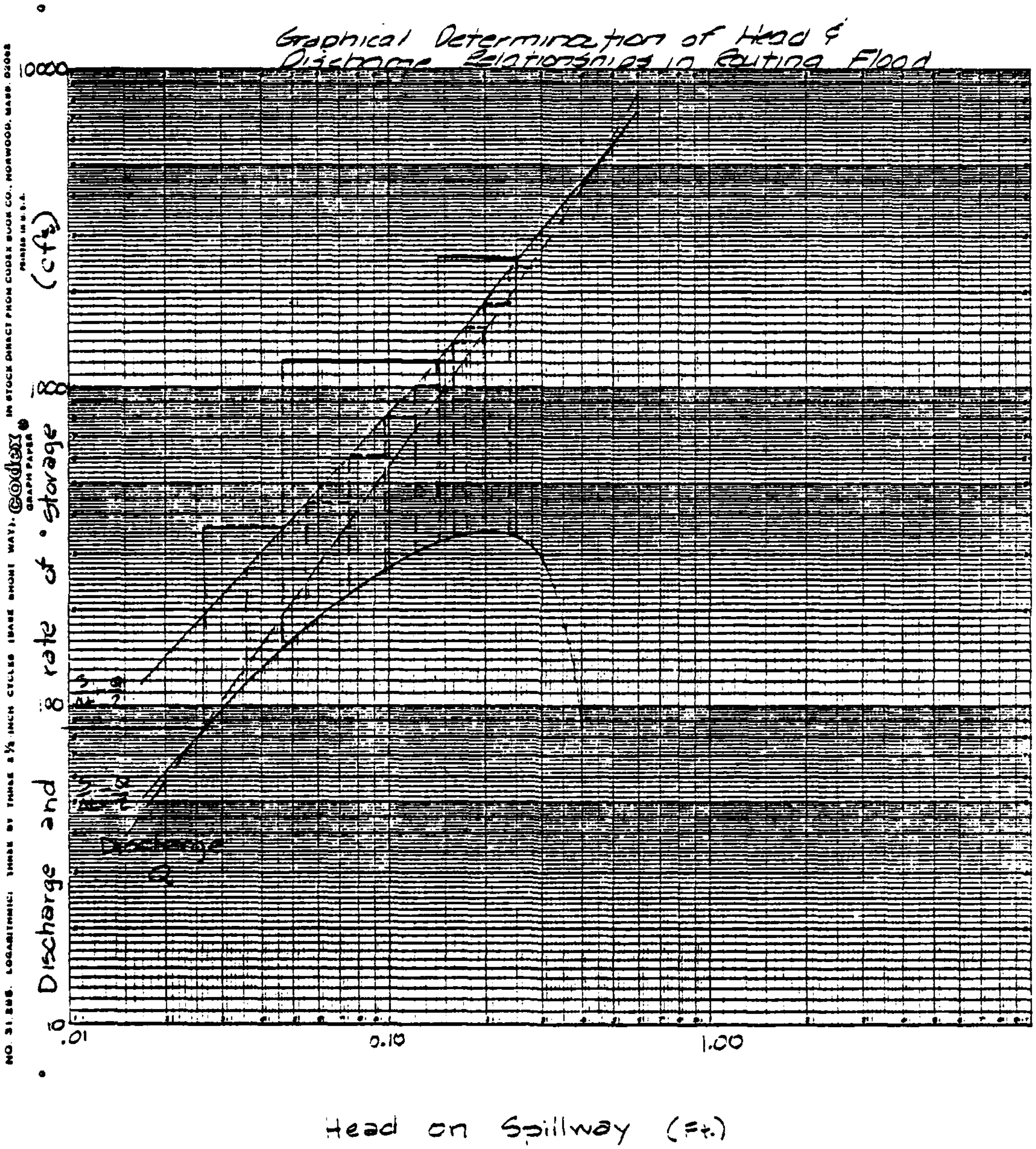

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TIME NUMBER (24 MIN. INTERVAL)	OBSERVED FLOW I (cfs)	AVERAGE INFLOW (cfs)	$\frac{S}{\Delta t} - \frac{Q}{2}$ AT BEGINN'G OF TIME INTERVAL	$\frac{S}{\Delta t} + \frac{Q}{2}$ AT END OF TIME INTERVAL	HEAD ON SPILLWAY (ft)	OUTFLOW Q (cfs)
0	0	—				
1	85	42			.0263	85
2	470	278	84	362	.0465	192
3	1660	1065	151	1216	.143	950
4	2840	2250	330	2580	.25	2075
5	2440	2640	340	2980	.28	2450
6	1685	2062	315	2377	.238	1910
7	1280	1482	345	1827	.197	1490
8	1110	1195	355	1550	.175	1240
9	950	1030	350	1380	.159	1090
10	775	862	340	1202	.142	930
11	585	680	328	1008	.121	740
12	420	502	305	807	.092	540
13	275	348	268	616	.075	375
14	145	210	225	435	.055	245
15	40	92	178	270	.036	118

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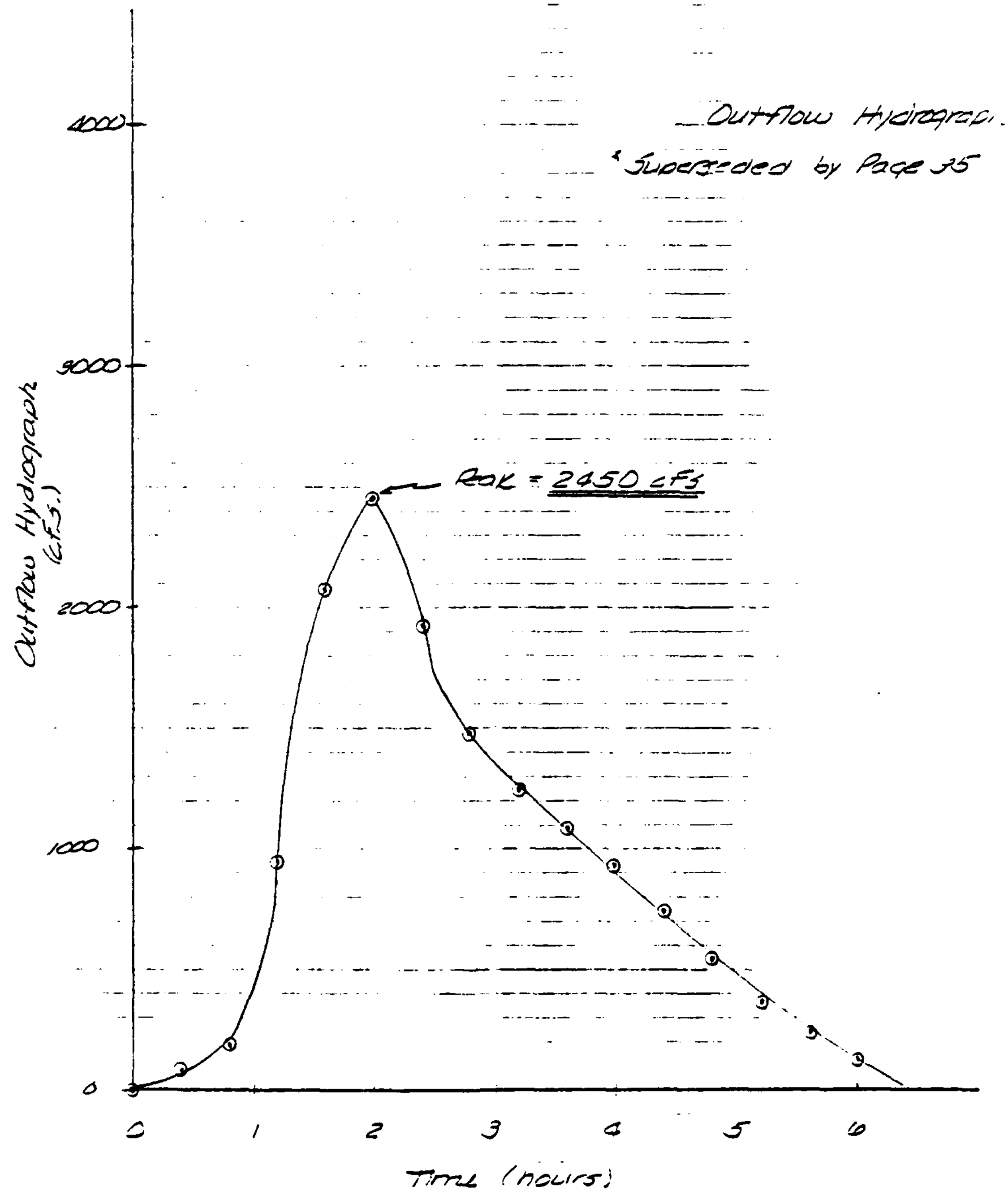

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Head on Pipe (ft)	Elev. of W.S.	Reservoir Area, A acres	Calc. Outflow O (cfs)	Calc. Storage S Acre-ft	$\frac{S}{\Delta t}$ cfs	$\frac{S}{\Delta t} - \frac{O}{2}$ cfs	$\frac{S}{\Delta t} + \frac{O}{2}$ cfs
38	56	165.0	54	0	0	0	0
39	57	170.8	55	167.9	5079	5052	5107
40	58	175.5	56	341	10315	10287	10343
41	59	180.6	56	519	15700	15672	15728
41.5	59.5	182.7	57	610	18453	18425	18482
41.6	59.6	183.2	567	628	18997	18713	19280
41.7	59.7	183.6	1506	646	19542	18789	20295
41.8	59.8	184.1	2740	664	20086	18716	21456
41.9	59.9	184.6	4270	682	20631	18496	22766
42.0	60	185.0	6037	700	21175	18157	24194

$\Delta t = 24$ min $= 1440$ sec

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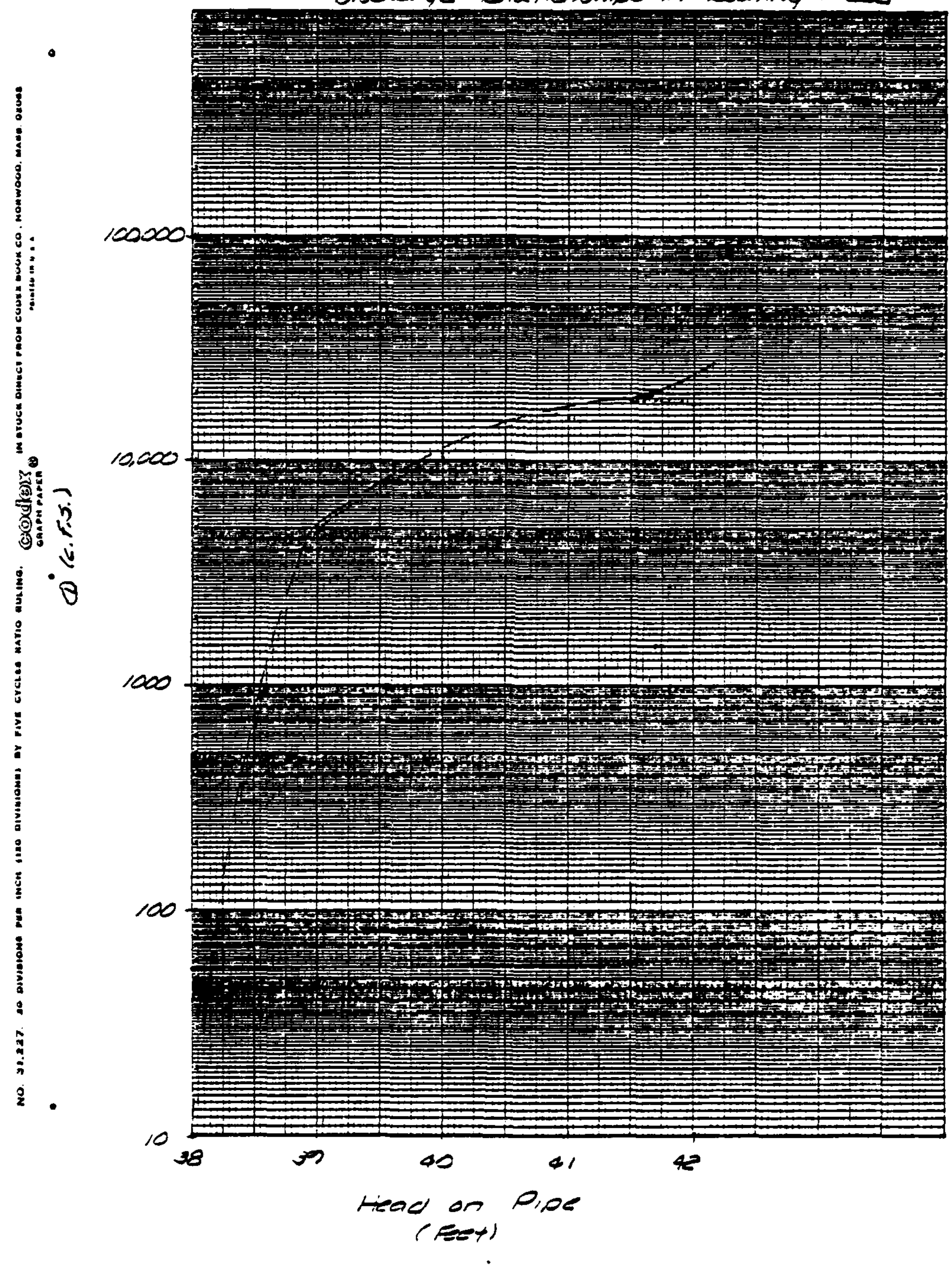
Graphical Determination of Head & Discharge Relationships in Routing Flood



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Time No.	Obs. Inflow (cfs)	Average Inflow (cfs)	$\frac{S}{\Delta t} - \frac{O}{2}$ (cfs)	$\frac{S}{\Delta t} + \frac{O}{2}$ (cfs)	Head on Pipe (u.s. invert)	Elevation (MSL) (ft)	Outflow (cfs)
0	0	—					
1	85	42			39.0	57	54
2	470	278	5052	5130	39.003	57.005	54
3	1660	1065	5076	6141	39.20	57.20	55
4	2840	2250	6087	8337	39.62	57.62	55
5	2440	2640	8282	10922	40.11	58.11	55
6	1685	2062	10867	12929	40.48	58.48	55
7	1280	1482	12874	14356	40.75	58.75	55
8	1110	1195	14301	15496	40.96	58.96	55
9	950	1030	15441	16471	41.14	59.14	55
10	775	862	16044	16907	41.21	59.21	55
11	585	680	16262	16942	41.22	59.22	56
12	420	502	16280	16782	41.19	59.19	55
13	275	348	16200	16548	41.15	59.15	55
14	145	210	16083	16293	41.10	59.10	55
15	40	92	15955	16047	41.06	59.06	55

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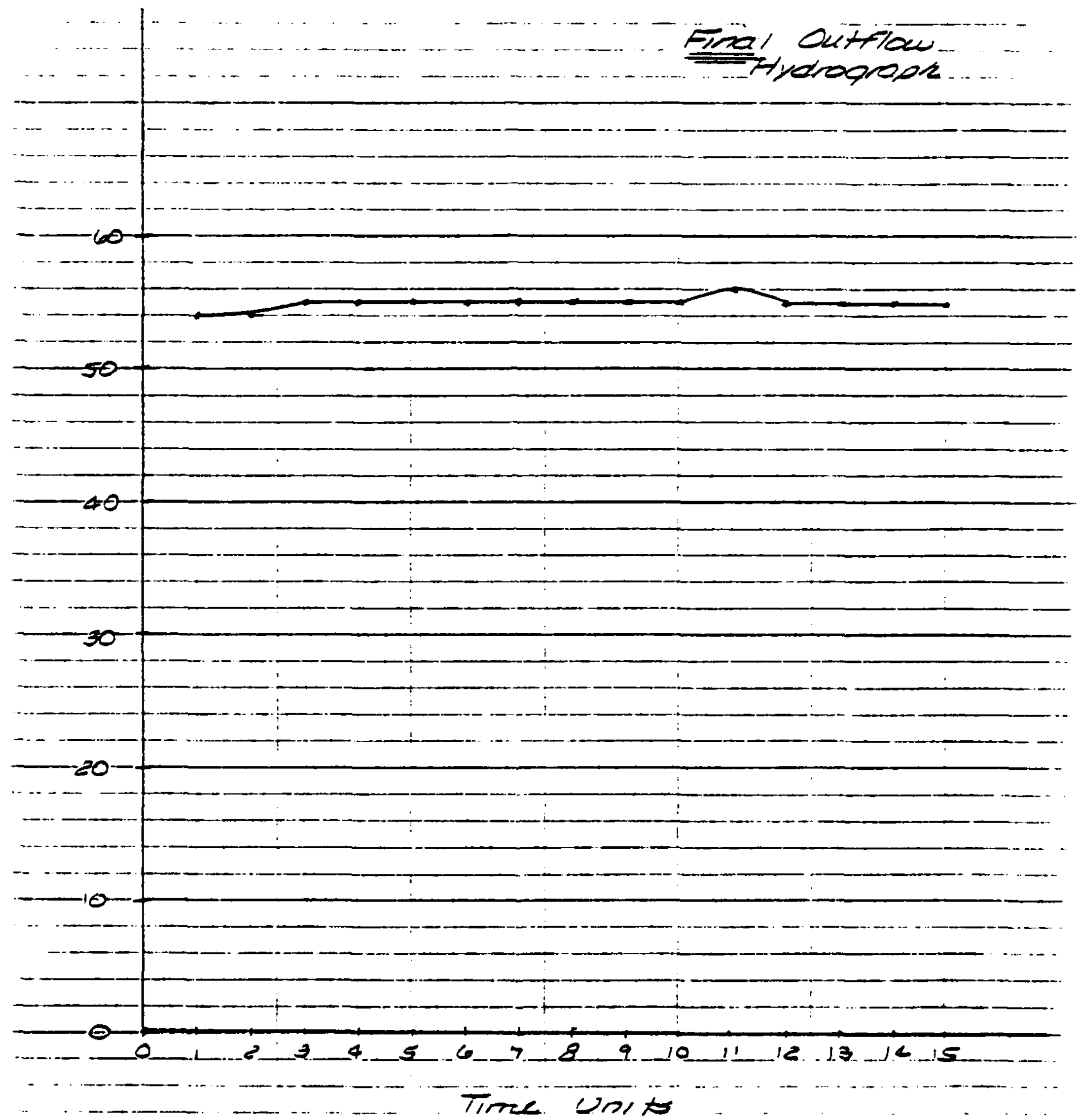
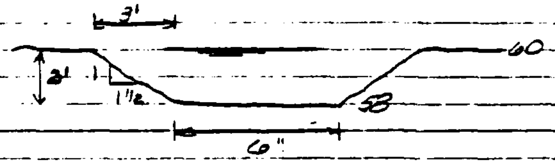


Capacities (continued)

Over-Flow Ditch

Cross Section Near North Street



$n = 0.025$ to 0.035

$Q = \frac{1.49}{n} R^{2/3} S^{1/2} A$

$A = 6 \times 2 + 2 \times 1/2 \times 3 \times 2 = 18.0$

$WP = 6 + 7.2 = 13.2$

$R = 1.364$

$Q = \frac{1.49}{.030} \times 1.364^{2/3} \times 18 \times S^{1/2}$

$Q = 1099.66\, S^{1/2}$

$S = .019048$

$_{50}Q = 151.8 \sim 152$ cfs

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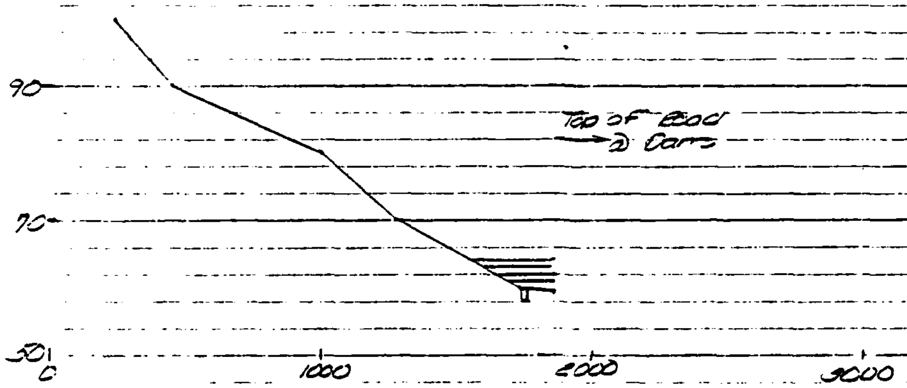
Typical Cross Section of Over Flow Ditch



Head	Elev.	Area	WP	R	S	Q
2'	60.0	18	13.2	1.364	.019098	152
3'	61.0	168	153	1.100	↓	1227
4'	62.0	353	200	1.765	↓	3535
5'	63.0	590	255	2.314	↓	7077
6'	64.0	875	310	2.823	↓	11985

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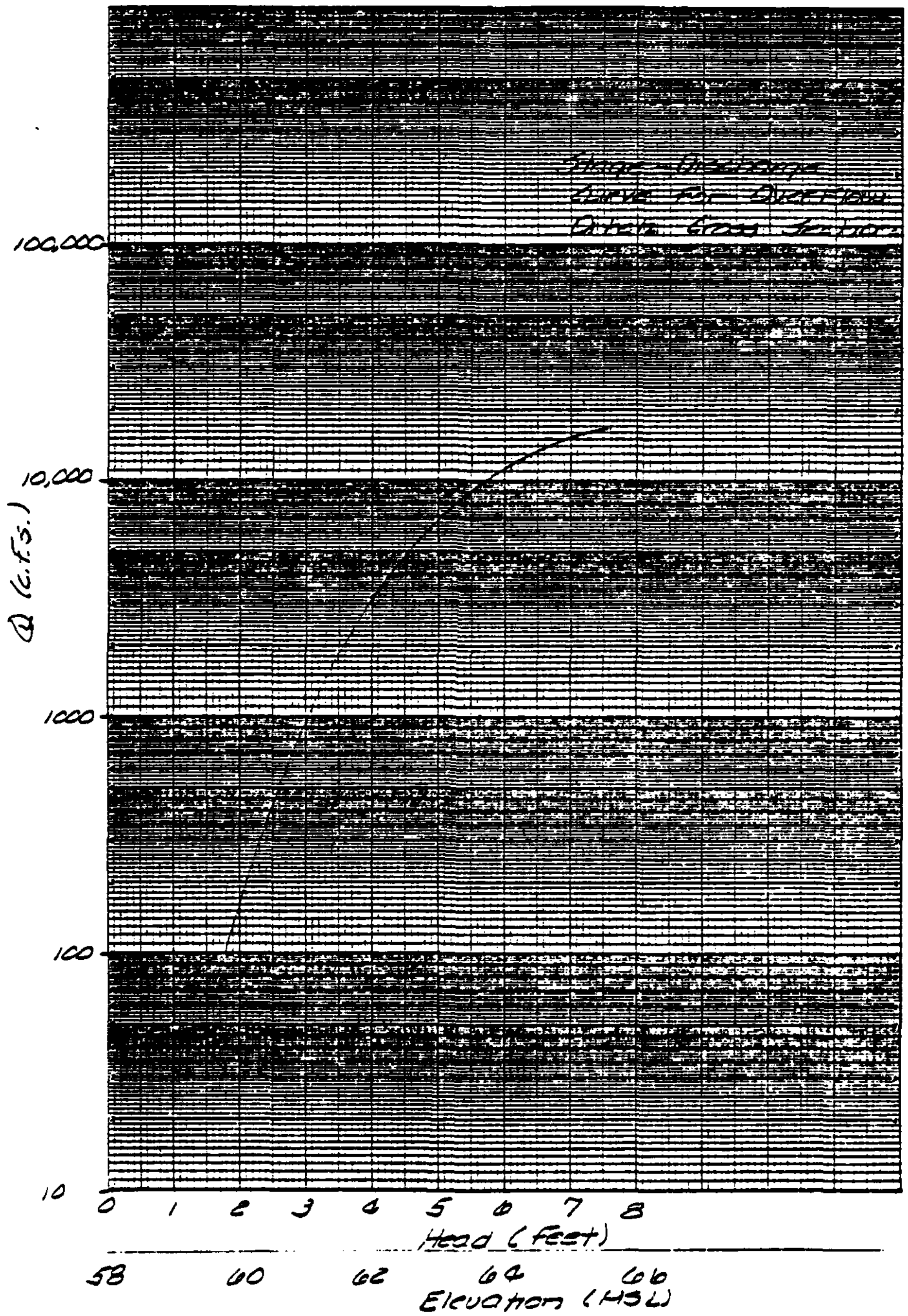

Capacities

Twin 36" R.C.

- upstream invert ≅ 52'
  downstream invert ≅ 51.5'
  length ≅ 60'
  slope ≅ .008333

$h_L = 2\ f\ \dfrac{LV^2}{D2g}$

$f = .0170$
$L = 60$ ft
$D = 3$ ft

$V = \dfrac{1.49}{n}\ R^{2/3}\ S^{1/2} = \dfrac{1.49}{.013}\left(\frac{3}{4}\right)^{.667}(.008333)^{1/2}$

$V = 8.64$ ft/s

$h_L = 2 \times \left(\dfrac{.0176 \times 60' \times (8.64\ \text{ft/s})^2}{3' \times 2 \times 32.2\ \text{ft/s}^2}\right) = 0.907'$ for 60' length of twin 36

$h_L = 15.15'/1000'$ for 2 pipes

$Q = 0.279\ (110)\ (3)^{2.63}\left(\dfrac{7.58}{1000}\right)^{.54}$

$Q = 37.5$ cfs for one 36" pipe

Q (for twin 36") = 77 cfs

At day of inspection, water surface in reservoir was at elevation 52.6 and twin 36" had approximately 6 inches of water in them so, at normal pond level the twin 36" would be full.

Road is @ elev. 60.0 ft., pond @ 56'

At wsel = 60', $Q = 2 \times .72 \times \frac{\pi 9}{4}\sqrt{64.4 \times 4} \cong 164$ cfs max
(assume no ponding)
out this isn't practical

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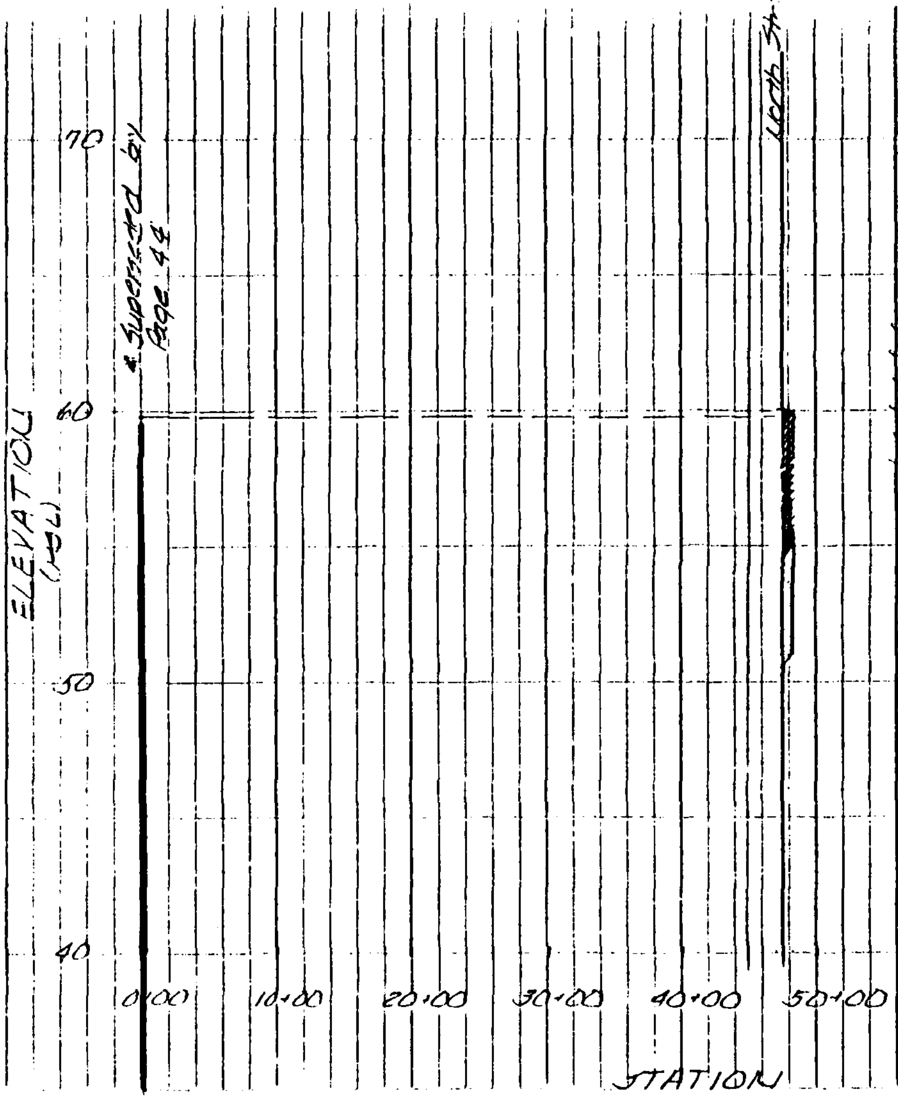

At PMF, peak WSE would reach 59.75. Stage Dis-charge curve [illegible] flow [illegible] at time of peak is the following.

Elev. DS	Elev. US	Q (cfs) (Total)
59.75	59.75	0
59.75	60.0	41
59.75	61.0	145 * start weir flow
59.75	62.0	352
59.75	63.0	560
59.75	64.0	1464

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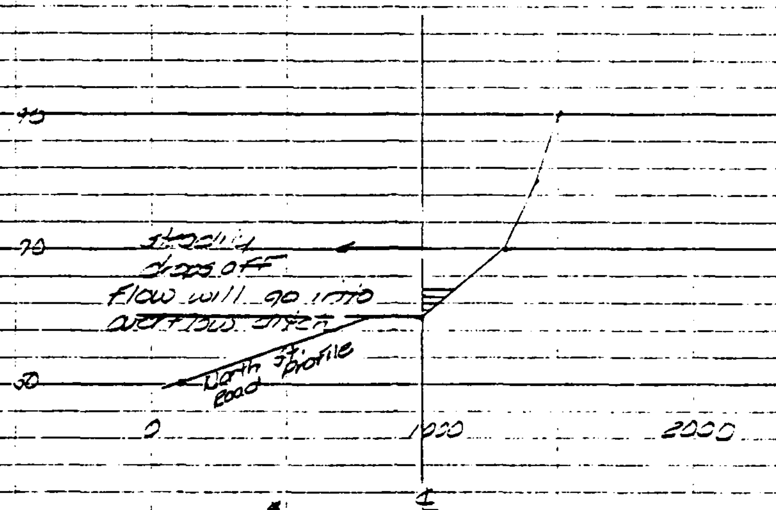
Cross Section of Twin 36" R.C.P. (Looking downstream)



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Elev.	Head on Rd.	Qculvert	Qweir
60.0	0	41	0
61.0	1.0	91	54
62.0	2.0	123	229
63.0	3.0	158	702
64.0	4.0	168	1390

Note: C=2.70 [illegible] coefficient for [illegible]

Flow from Upstream section of Watershed

≅ 4130 cfs

CAPACITIES

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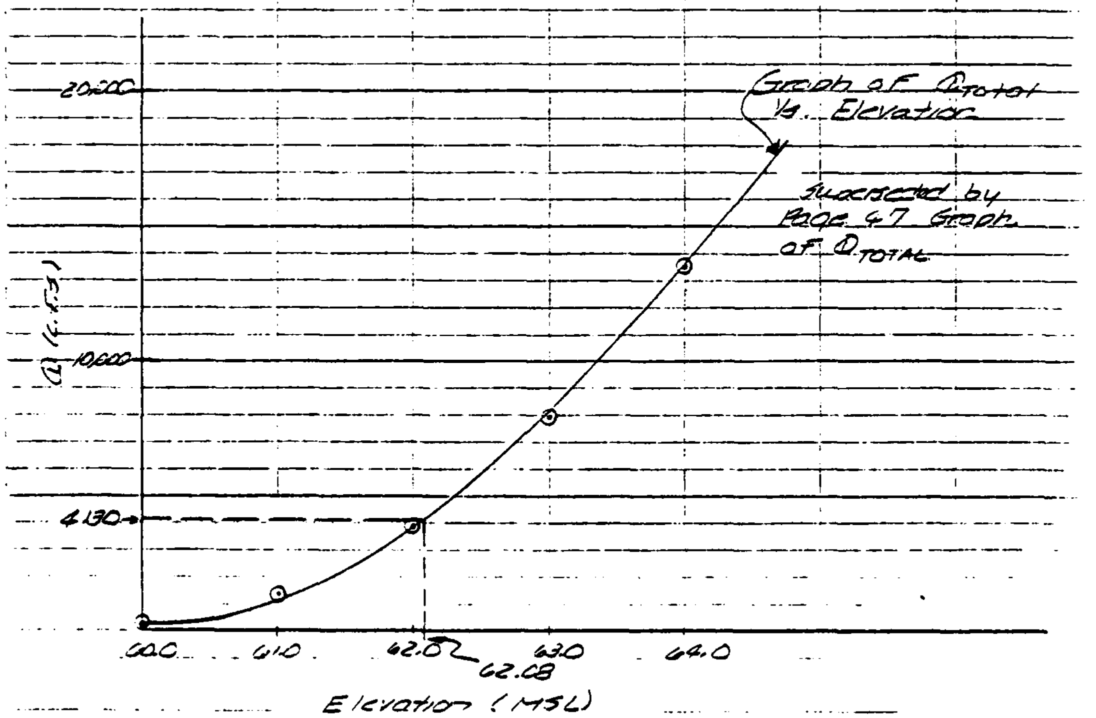
Elev.	Q (twin 36")	Q (overflow ditch)	
60.0	41	152	193
61.0	145	1227	1372
62.0	352	3535	3887
63.0	860	7077	7937
64.0	1464	11985	13449



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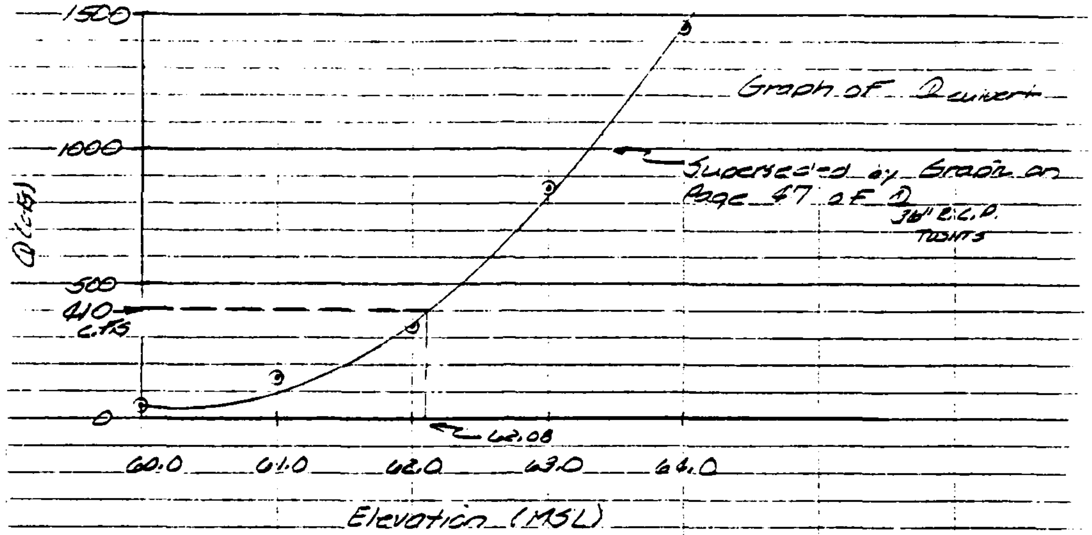

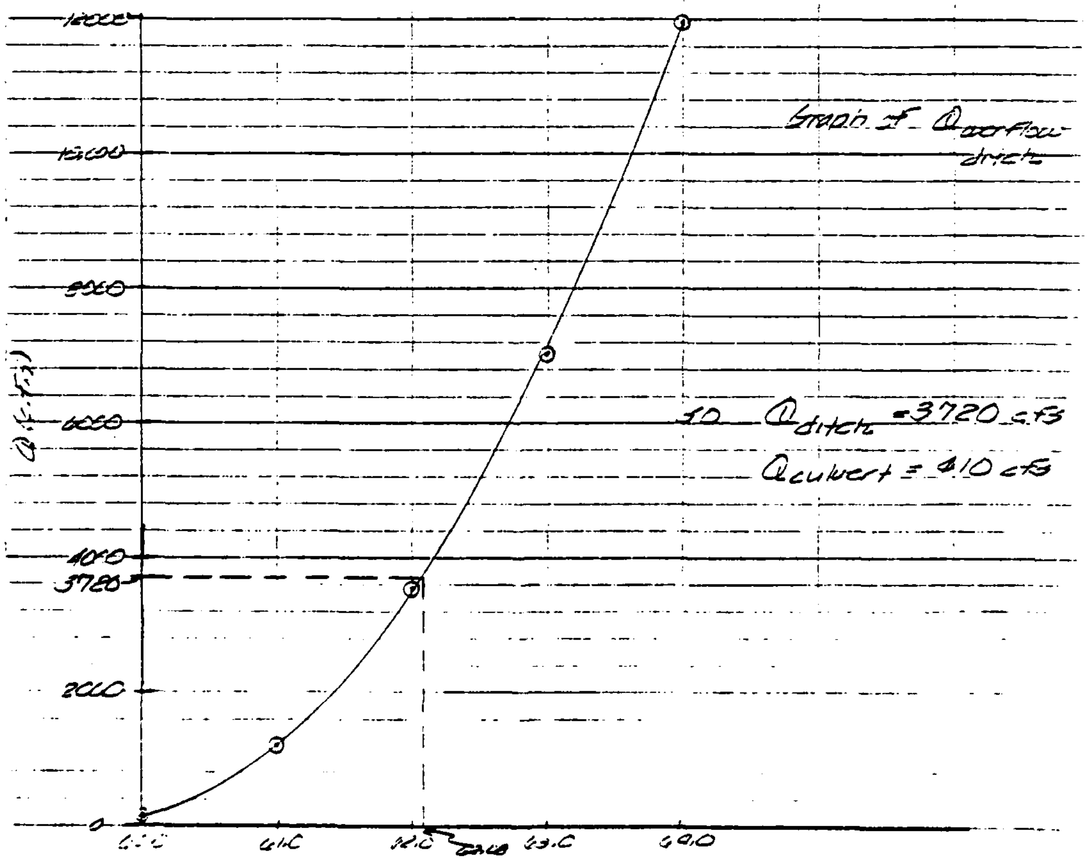

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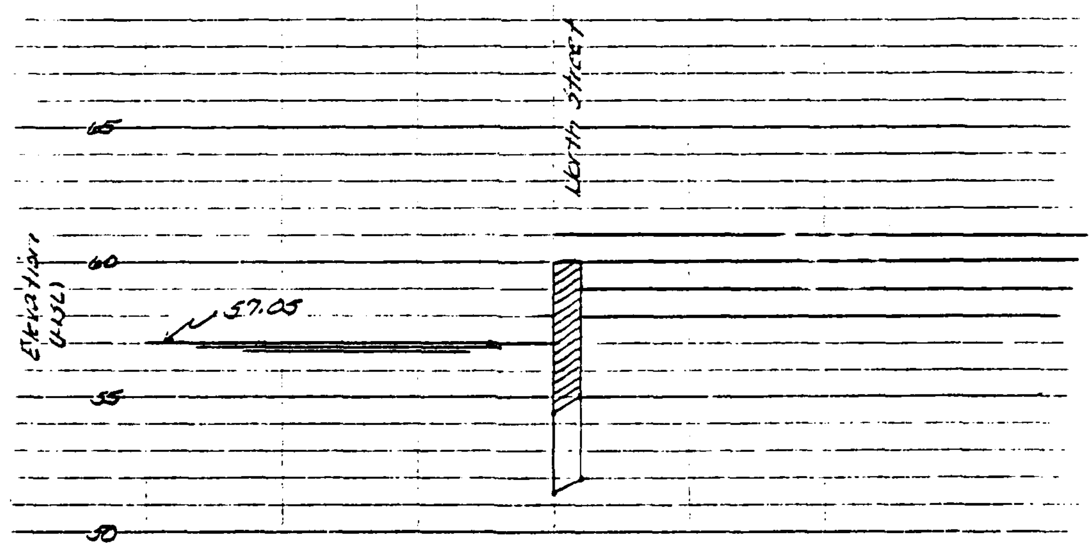
Because the flows from each of the drainage areas do not peak at the same time, the following is an exercise using the peak flow of the northern drainage area (4131 c.f.s) and the flow which occurs at reservoir from the southern drainage area at the time of the northern D.A.'s peak.

Northern D.A. : 4131 c.f.s. at t_p = 4 hours

Southern D.A : 770 c.f.s. at t = 4 hours from start of the storm.

At 770 c.f.s., Water Surface Elevation in Reservoir = 57.05'



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Stage - Discharge Relationships for Twin 36" Culvert

Elev. U.S. W.S.E	Head on Road	Head on Culvert	Q culvert	Q weir
57.05	0	0	0	0
58	0	.95	80	0
59	0	1.95	114	0
60	0	2.95	140	0
61	1	3.95	162	54
62	2	4.95	182	229
63	3	5.95	199	702
64	4	6.95	215	1290

NOTES

Weir Coefficient "C" ≅ 2.70 for road

Pressure Flow Coefficient "C" ≅ 0.72 for culvert

$$Q_{\substack{pressure \\ flow}} = CA\sqrt{2gh}$$

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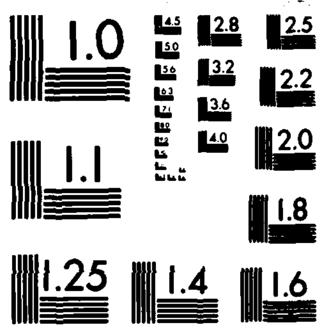

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

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Flow From Northern Section of Watershed
≅ 4131 c.f.s.

CAPACITIES

Elev. (msl)	Q (twin 36")	Q (overflow ditch)	Q (TOTAL)
57.05	0	0	0
58	80	0	0
59	114	44	158
60	140	152	292
61	216	1227	1443
62	411	3535	3946
63	901	7077	7978
64	1511	11985	13496

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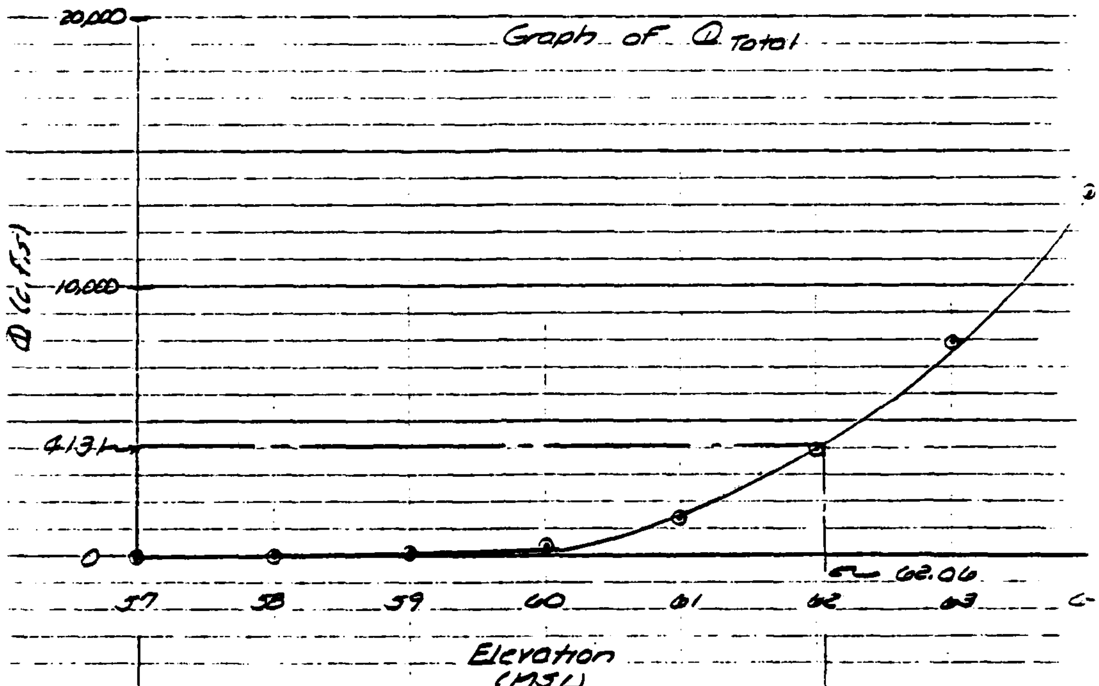
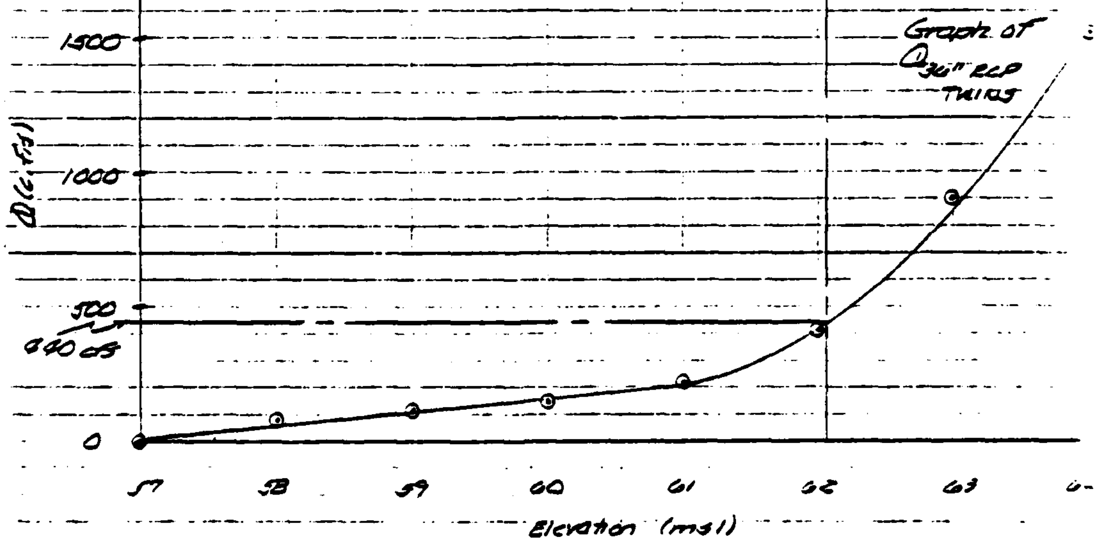




Therefore, approximately 440 c.f.s goes through twin 36" R.C.P. (and over North St.) and approximately 3720 cfs goes out the overflow ditch.

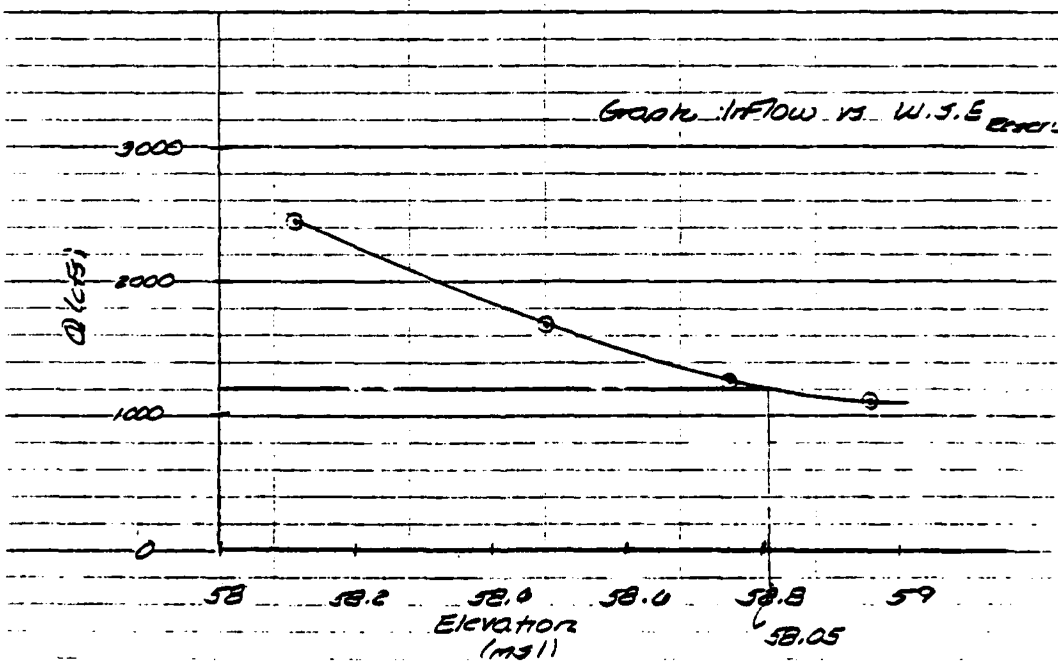
So, at t = 4 hours, Reservoir Inflow = 440 cfs (U.O.A.)

+ 770 cfs (3.0 A)

= 1210 c.f.s total

Approximate Out Flow after Routing = 55 c.f.s.

Graph: Inflow vs. W.S.E Reservoir



$Q_{outflow}$ = 55 cfs at t = 4 hours

If you consider using T = 11.65 hours (the time when the Southern Drainage Area Flow peaks) and 3790 cfs as the peak of the S.D.A., the flow from the Northern Drainage Area would equal approximately 16 cfs (at T = 11.65 hrs).

Considering the large reservoir surface area, the 16 cfs will not cause a significant increase in the reservoir's water surface elevation.

Therefore, the final reservoir elevation and corresponding outflow is 59.22 ft (above msl) and 56 c.f.s.

APPENDIX E
INFORMATION CONTAINED IN
THE NATIONAL INVENTORY OF DAMS

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